HYDRO - ELECTRIC DEVELOPMENT ON THE WEST BRANCH OF THE PENOBSCOT RIVER, MAINE

BY

J. G. COONEY

H. N. SIMPSON

R. W. KINNALLY

ARMOUR INSTITUTE OF TECHNOLOGY

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PROPOSED HYDROELECTRIC DEVELOPMENT

ON THE WEST BRANCH OF THE

PENOBSCOT RIVER, MAINE.

A THESIS

PRESENTED BY

James G. Cooney,

Herman N. Simpson,

Raymond W. Kinnally.

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CIVIL ENGINEERING

APPROVED:

Alfred E. Phillips
PROFESSOR OF CIVIL ENGINEERING

A. M. Raymond
DEAN OF ENGINEERING STUDIES

L. C. Morrison
DEAN OF CULTURAL STUDIES

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PROPOSED HYDROELECTRIC DEVELOPMENT ON THE WEST BRANCH OF THE PENOBSCOT RIVER, MAINE.

Part 1.

LOCATION

A. Geographical

There are two main branches to the Penobscot River,- the West Branch and the East Branch. The West Branch rises in the mountainous region near the Canadian boundary and flows in a southeasterly direction for more than 100 miles to its junction with the East Branch at Medway. From this point to the mouth of the river comprises another 100 miles. The river is therefore 200 miles long.

About 50 miles from its source, the West Branch flows into Lake Chesuncook, thence, almost directly into Lake Ripogenus, and finally with a fall of 214 feet in 2.4 miles, it reaches a calm, quiet stretch of water called Big Eddy. In the stretch of 2.4 miles from Lake Ripogenus to Big Eddy the river is a torrent, flowing through ledges with almost vertical sides, 40 to 75 feet high.

In many places it is very narrow and affords excellent dam sites. The drainage area at the outlet of Ripogenus Lake, is 1410 square miles. Under natural conditions this would provide an extremely low-water flow of 218 second-feet, which would give, under a fall of 214 feet, nearly 10000 gross horse-power. Under fully developed storage, this could be developed to 40000 horse-power. It is this latter proposition that the authors of this thesis have chosen to develop.

B. Transportation.

The Bangor & Aroostook Railroad connects with the Maine Central at Bangor, and extends from this city to the extreme northern part of the state. A branch of this road, also extends from a point just below the junction of the two branches of the river, along the West Branch to the foot of Lake Chesuncook

and thence on to the St. John River. From the foot of the former Lake to the foot of Lake Ripogenus is about 2¼ miles, and approximately 5 miles to Big Eddy. It is thus seen that the power site is not without suitable transportation facilities.

C. Industries.

The great forests of the basin of the West Branch, and the water power afforded by the river has had an important influence on the industries of this region. Long and short lumber, including shingles, boards, lathes, etc., and pulp, paper, and woolen goods are extensively manufactured. These industries are still in their infancy, and as cheap power becomes available, through the intelligent development of the unharnessed energy of the Penobscot River and its numerous tributaries, they will

expand and increase manifold. As an example, the manufacture of wood pulp has during the last ten years become one of the leading industries of this area. Conditions for this industry are ideal, and will not reach its maximum development for many years to come.

D. Cities.

Within the Penobscot River basin are over thirty towns with a combined population of approximately 100,000. The city of Bangor to which the proposed transmission line of the power plant is to extend is the largest of these cities and has a population of 25,000. It is about 86 miles from Big Eddy measured along the proposed route of the transmission line. Although the city of Bangor is favorably situated with regard to raw material, it is not so well situated with

regard to cheap power. Hence the assured success of a properly managed and constructed power plant of large capacity within a reasonable distance of the city.

PROPOSED HYDROELECTRIC DEVELOPMENT ON THE WEST BRANCH OF THE PENOBSCOT RIVER, MAINE.

Part 11.

HYDRAULIC.

A. GENERAL PLAN.

It will be seen from the perusal of plate 1. that the storage basin of the proposed plant will consist of the two lakes "Chesuncook" and "Ripogenus", together with their numerous smaller adjacent lakes. In their present natural state the storage capacity of the above lakes is not sufficient for a continuous development of 40,000 horse-power. It is therefore proposed to build a dam at the lower end of Lake Ripogenus which will serve two purposes:-

1. The obvious and fundamental function of storing the spring floods for use during the dry summer and autumn months.

2. There being a continuous and sufficient flow of water into the lakes,- the present level of Lake Ripogenus(883.3) will be raised some 50 feet, which is 7 feet higher than the present mean elevation of Lake Chesuncook(927.8); so that

the two lakes will have finally equal elevations of 934.6; thereby increasing their combined storage capacity from 13.9 billion cu. ft. to 23.8 billion cu. ft., which is shown by calculations in the succeeding pages to be sufficient to keep an average load of 31,400 horse-power plant running 24 hours a day the year around.

That it is topographically possible to raise the level of the two lakes to the above proposed elevation is shown by the location of the contours on plate 1. From government reports it has been ascertained that the property damage resulting from the above change would be slight.

The elevation of "Big Eddy", the location of which has been mentioned as being 2.4 miles below the end of Lake Ripogenus, is 668.0. The total maximum head made available by the construction of the dam discussed in the preceeding paragraph will then be 934.6 - 668.0 = 266.6

feet. To take advantage of this head, a pipe line would be constructed from the dam to Big Eddy where the proposed power house is to be located, a distance of about 12,500 feet.

B. Available Power.

Precipitation:-

The earlier measurements of precipitation in the upper regions of the Penobscot basin are fragmentary. It was not until comparatively recent years that precipitation stations were established in order to gather reliable data as to the actual amount of rainfall. Since the topography of the drainage basin varies considerably, the precipitation records at any one point cannot be taken as a criterion for the whole or any other part of the region.

During the winter months heavy snows often accumulate to a depth of several feet, especially in the wooded districts. With the advent of warm weather, the melting of the snow and the heavy spring rains causes a large amount of water to be liberated in short periods of time, during which floods generally occur. The rainfall in the Penobscot basin is not as a rule excessive, except in the spring, and there are few floods in the streams.

The average yearly precipitation for a period of five years from 1905-1909 inclusive, at the foot of Lake Chesuncook was 35.56 inches, the minimum being 31.09 inches in 1908 , and the maximum 40.74 inches in 1909.

Discharge:-

The character of the Penobscot River basin presents a most favorable condition for

accurate determination of discharge. The stream bed, as a rule, is composed of rock, and is not subject to change by deposits of sediment and loose material.

No records for the stream flow of the West Branch of the Penobscot River, at the outlet of Lake Ripogenus are available; but very accurate data is available however, for a point about twenty-five miles further downstream, at Millinocket. The discharge of the river at the Millinocket Mill of the Great Northern Paper Co. has been computed and recorded daily since 1901. The available records for a period of ten years from 1901-1909 inclusive has been published in the Water Supply Paper #279 of the U. S. G. S. It is this data, corrected for difference of drainage areas at Millinocket and the foot of

Lake Ripogenus, which has been used in all calculations following. The correction factor used is equal to the ratio of the drainage areas at the foot of Lake Ripogenus and at Millinocket, which is equal to 1410/1880 or 0.75. No appreciable error results from applying this correction factor, as the land drained by the river above the two points in question is identical in character.

The Records were obtained by considering the flow through the wheels of the mill and the flow over the dam, and the water used from time to time by log sluices.

The maximum daily discharge occurred April 1 and 2,1903, and equalled 18,200 second feet. The minimum weekly discharge equalled 218 second feet and occurred from Jan. 28 to

Feb. 3,1904.

Runoff:-

The following table shows the mean ratios of run-off to precipitation, for the West Branch of the Penobscot River at Chesuncook Lake Dam.

Table 1.

	Precip. in in.	Runoff in in.	Mean ratio of runoff to precip. 1901-1909
Jan.	3.22	0.69	0.21
Feb.	2.25	0.51-	0.23
Mar.	4.05	1.03	0.25
Apr.	2.96	4.66	1.58
May	2.75	7.42	2.70
June	3.18	2.75	0.86
July	3.24	1.20	0.37
Aug.	3.36	0.76	0.23
Sept.	3.97	0.56	0.14
Oct.	3.37	1.16	0.34
Nov.	2.86	1.22	0.43
Dec.	3.72	0.94	0.25
Year	38.90	22.90	0.59

As has been previously stated, it is proposed to erect a dam at the outlet of Lake Ripogenus, which will raise the level of this lake and also of Lake Chesuncook up to an elevation of 934.6. This will require a dam of about 70 feet in height.

Lake Ripogenus is about twenty-five miles above Millinocket, and is more like an enlargement of the river than a lake, being about two and a half miles long, and from one - half to three-quarters mile wide. Its banks are high except in the upper portion where Harrington Stream enters; here they are low and marshy for a considerable distance inland.

At present there is a dam at the outlet of the Lake, used to obtain a flush to carry logs through Ripogenus Gorge to Big Eddy, a distance of 2.4 miles in which the fall is 215 feet. The area of the lake at elevation 883.3 is 1.07 square

miles and at elevation 889, the area is 1.27 square miles. The total present capacity is 301,100,000 cubic feet. This will be increased 5,000,000,000 cubic feet.

Separated from the head of Lake Ripogenus by only 0.57 miles of river is Lake Chesuncook. This lake is about 17.7 miles long, and from three-quarters to a mile wide, and has a fairly regular shoreline with high banks.

At the outlet of Lake Chesuncook is a timber dam, also used to obtain a flush for logs, between this lake and Lake Ripogenus. The elevation of Chesuncook when drawn down to the deep sill gates is 908.6 mean sea level. When at its full height it has an elevation of 930.6 feet, and an area of 32.1 square miles. The present storage capacity of 13.6 billion cubic feet is to be increased by 4.0 billion cubic feet, giving a total of 17.6 billion cubic feet.

The drainage area at the head of the lake is 825 square miles, and at the outlet 1330 square miles.

The following table gives the area and capacity of the storage lake for different elevations.

Table 2.

Elev. above sea level	Area of Water Surface	Total Cap. above elevation 879.0 Million Cu. Ft.
889	1.07	301
890	1.27	335
892	1.30	515
894	1.32	696
896	1.34	876
898	1.38	1,056
900	1.45	1,239
902	1.58	1,419
904	1.69	1,500

Table 2.(Con.)

Elev. above sea level	Area of Water Surface	Total Cap. above elevation 879.0 Million Cu. Ft.	
906	1.82	1,780	
908	1.88	1,960	
910	1.95	2,140	
912	20.1	3,120	
914	20.3	5,194	
916	20.5	6,478	
918	21.7	7,841	
920	24.2	9,345	
922	27.6	11,038.5	
924	30.0	12,862	
926	31.8	14,786	
928	33.6	16,809	
930	34.1	18,863	
932	37.9	21,126	
934	37.9	23,390	
934.6	37.9	23,800	Crest of proposed spillway

Quantity of Water Necessary for Plants:-

The amount of water to be used, assuming that an average load of 31,400 horsepower is to be carried is determined by the formula

$$Q = \text{hp.} \times 550/62.5 \times H(\text{Feet})$$

Assuming H as 240 feet and hp. as 31,400,

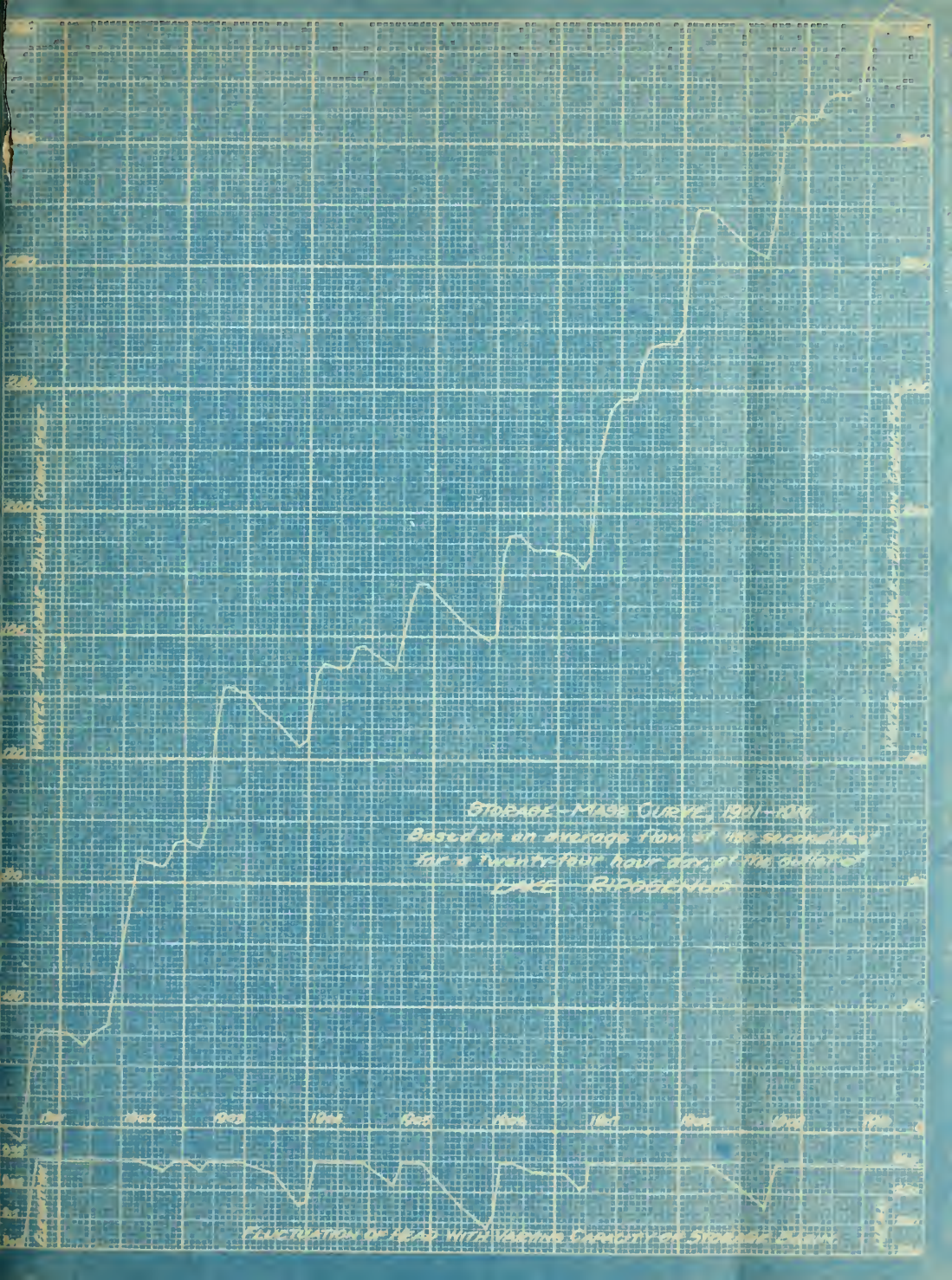
$$Q = 31{,}400 \times 550/62.5 \times 240$$

$$= 1150 \text{ cu. ft./ sec.}$$

This value of 1150 cu. ft. per sec. has been used ot obtain the data from which the following mass curve has been drawn.

The features of this diagram are as follows:

(a) For the interval of time between any two dates represented on the axis of abscissa, the surplus or deficiency, above 1150 sec. ft., is obtained by subtracting the ordinate correspond-


Storage Mass Curve, 1921 Flow
Based on an average flow of the Sacramento River for twenty-four hour days at Red Bluff
Daily Discharge
Flow Station No. 1923 Mississippi Research Board

ing to the earlier date from the ordinate corresponding to a later date. If this difference is positive it represents a surplus; if negative it represents a deficiency.

(b) The crests and hollows of the curve indicate those instants of time when supply and demand are equal.

(c) If a horizontal line is drawn from any of the low points of the curve back to a rising line, the maximum ordinate scaled from the horizontal line to the curve will show the amount of water that would have to be stored to provide for the assumed flow during the period of drought covered by the horizontal line.

(d) The period during which this ordinate occurs is therefore the critical one, and all surplus of supply over demand during parts of this period must be stored to meet the deficiency during the remainder of it.

The mass curve was plotted from a table computed on the basis of the mean monthly discharge of the river for a period of ten years,(1901-1909 inc;) the first two years and the last two of which are given in table 3.

The storage mass curve shows that the period which includes the maximum ordinate extends from July 1903 to April 1904, and corresponds to about 19,000,000,000 cubic feet. This amount of water is then necessary to meet the demand during dry period on record.

The greatest fluctuation of head which is produced by the drawing down of the storage, is about twenty feet or about 8.5 percent of the total head.

No account has been taken of the water

Table #3.

Mass Curve of W. Br. Penobscot River at Foot of Lake Ripogenus.

	Disc. of R. (cor for stor)	Surplus (+) or deficit (-) if Q=1150 1901.		Water available for storage. Billion cu. ft.
Jan.	600	- 550	- 1.445	- 1.445
Feb.	630	- 520	- 1.368	- 2.813
Mar.	545	- 605	- 1.590	- 4.403
Apr.	9220	+8070	+21.200	+16.797
May	6120	+4970	+13.050	+20.847
June	1800	+ 650	+ 1.710	+31.557
July	1050	- 100	- 0.263	+31.294
Aug.	1380	+ 230	+ 0.605	+31.899
Sept.	530	= 620	- 1.630	+30.269
Oct.	307	- 847	- 2.225	+28.044
Nov.	480	- 670	- 1.760	+26.284
Dec.	2350	+1250	+ 3.280	+29.564

Table #3(Con.)

1902

Jan.	1950	+ 800	+ 2.100	+31.664
Feb.	1640	+ 490	+ 1.287	+32.951
Mar.	3500	+2350	+ 6.170	+39.121
Apr.	10950	+9870	+25.900	+55.021
May	7650	+6500	+17.100	+72.121
June	7050	+5900	+15.500	+87.621
July	645	- 505	- 1.328	+86.303
Aug.	860	- 290	- 0.761	+85.542
Sept.	1050	- 100	- 0.263	+85.279
Oct.	2230	+1080	+ 2.840	+88.119
Nov.	2770	+1620	+ 4.250	+92.369
Dec.	1725	+ 575	+ 1.380	+93.749

Table #3(Con.)

(1903-1908 inc. omitted)

1909

Jan.	607½	- 542½	- 1.425	+285.764
Feb.	563	- 587	- 1.540	+284.224
Mar.	795	- 355	- 0.932	+283.292
Apr.	6185	+5035	+13.220	+296.512
May.	11400	+10250	+26.90	+323.412
June	2700	+1650	+ 4.330	+327.742
July	1725	+ 575	+ 1.510	+329.252
Aug.	697½	- 452½	- 1.190	+328.062
Sept.	1455	+ 305	+ 0.801	+328.863
Oct.	3180	+2030	+ 5.330	+334.193
Nov.	1926	+ 776	+ 2.040	+336.233
Dec.	1431	+ 281	+ 0.737	+337.070

Table #3(Con.)

1910

Jan.	1071	- 79	- 0.207	+ 336.763
Feb.	1250	+ 65	+ 0.171	+ 336.934
Mar.	1187	- 63	- 0.165	+ 336.769
Apr.	6990	+5840	+15.320	+ 353.089
May	4740	+3590	+ 9.430	+ 361.519
June	3600	+2450	+ 6.430	+ 367.949
July	1680	+ 530	+ 1.390	+ 369.339
Aug.	787	- 363	- 0.954	+ 368.385
Sept.	300	- 850	- 2.230	+ 366.155
Oct.	263	- 887	- 2.230	+ 363.825
Nov.	787	- 363	- 0.954	+ 362.971
Dec.	488	- 662	- 1.740	+ 361.231

required for log-driving, but by efficient methods this can be reducted to a minimum, and will have little effect on the storage as a whole.

C. Design.

1. Dam.

The dam is to be located at the outlet of Lake Ripogenus, at a point where the distance between the 940.0 foot contours on both sides of the river is the shortest. The dam will have a position perpendicular to the flow of the river, and will be composed of a central spillway section, which on the north will be continued as an earth dam. On the south, the spillway section will join retaining and intake sections, which will in turn join an earth dam for the remaining distance.

The spillway section was designed to discharge a flow of 20 second-feet per square mile

of drainage area, or a total of 1410 x 20, or 28,000 second-feet.

Using the Francis weir formula

$$Q = 3.33 \times H^{3/2} L$$

and assuming L = 600 feet, as occurring between stations 2.+ 0 and 8.+0(see plate 2), as measured from the south bank of the river, we obtain after substituting in the formula

$$H^{3/2} = Q/3.33 L$$

$$= 28,000/3.33 \times 600$$

$$= 14.1$$

$$H = 5.8 \text{ ft.}$$

H was made 5.4 ft., so as to bring the portion of the dam adjoining the spillway section up to elevation 940.0.

The largest section of the spillway was designed first; all other sections have been derived from this section by raising the level of the base to its proper height on the section. The section as designed is shown on

plate #3.

Design:-

Weight of concrete 150# per cubic foot.

Calculations are based on a lineal length of dam of 1 foot.

Weight of concrete in a section one foot long =

$$W_c = 150\ (2.5 \times 10 \times 22.5) \times 70.6$$

$$= 370{,}500.$$

W_c acts at a distance from Y-Y =

$$\frac{70.6 \times 22.5 \times 20 - 70.6 \times 6.6}{70.6\ (2.5 + 10 + 22.5)} = 12.7 \text{ Ft.}$$

The pressure of the water above the spillway =

$$P_2 = 10.0 \times 5.4 \times 62.5 = 3375.$$

The pressure of the water behind the dam =

$$P_1 + (5.4 + 35.3) \times 62.5 \times 70.6 = 179{,}500\ \#.$$

P_1 acts at a distance from the base =

$$(70.6 + 5.4) - (35.3 + 5.4 + 70.6^2/12 \times 40.7)$$

$$= 25.05 \text{ Ft.}$$

To find the point of application of the resultant, that is where the resultant cuts the base take moments of all the forces about the toe.

$$F = \frac{370,500 \times 37.3 + 3375 \times 50 - 179,500 \times 25.05}{370,500 + 3375}$$

$$= 25.2 \text{ ft.}$$

Since the base is 60 feet in length, the middle third would be 20 feet from the toe, so that the resultant pressure with a full quantity of water behind the dam falls well within the middle third.

Then there is no water behind the dam, the point of application of the resultant pressure is 37.3 feet from the toe. This is also within the middle third so that the spillway is stable under all conditions.

Pressures at heel and toe of the base.

$$P(\text{toe}) = 373,875/60 + 6 \times 4.8 \times 373,875/3600$$

$$= 9,225 \text{ \# per sq. ft.}$$

$$P(\text{heel}) = 373{,}875/60 - \frac{6 \times 4.8 \times 373{,}875}{3600}$$

$$= 3{,}225\ \#.$$

P(toe, when there is no water behind the dam) =

$$\frac{370{,}500}{60} - \frac{7.3 \times 6 \times 370{,}500}{3600} = 10{,}680\ \#.$$

Coefficient of stability against sliding=

$$\frac{179{,}500}{370{,}500} = 0.478$$

This will be increased by the use of a key, which will also help to cut off seepage under the dam.

The design of the earth dam and intake will not be treated in detail. The design of the retaining wall follows. The various sections of the spillway, earth dam, and retaining wall are shown on plates 2, 3 and 4.

Design of Retaining Wall for Earth Dam(Plate 4):-

The weight of earth is assumed as 120 lbs. per cubic foot.

The weight per lineal foot of wall =

$150(23 \times 1.16 + 7 \times 2 + 5 \times 1\frac{1}{2}) = 7500$ #.

To find center of gravity of concrete, take moments about X X.

$$\frac{7 \times 2 \times 4.08 - 3.08 \times 6 - 2.5 \times 2.24}{47.0} = 1 \text{ ft. from X X.}$$

Weight of earth over right cantilever,

$7 \times 21 \times 120 = 17,650.$

To find the c. g. of vertical forces, take moments about XX,

$$x = \frac{7 \times 21 \times 4.08 + 47 \times 1}{747 + 47} = 3.34 \text{ from X X.}$$

Horizontal component of earth pressures assume that H = 1/3 V.

$H = 21 \times 120/3 \times \frac{1}{2} \times 21 = 8800$ #.

$R = (17,650 + 7500^2 + 8800^2)^{\frac{1}{2}} = 26,650$ #.

R cuts base at $9 \times 8800/25,150 = 3.15$ from c. g. of vertical forces.

$\tan.\alpha = 8800/25,150 = 0.351.$

R cuts base at a distance from the toe equal to,

5.58 + 3.34 - 3.15 = 5.67 ft.

length of base = 13.16 ft.,

center of base = 6.58 ft.

middle third 4.29 ft., therefore eccentricity of load = 1.91 ft.

Design of vertical slab:-

Counterforts spaced 12 ' - o" c. to c.

The vertical wall will be designed as a continuous beam placed on edge.

The maximum bending moment will occur at the base.

$w = p = 1/3 \times 21 \times 120 = 840$ lbs. per sq. ft.

$W = 12 \times 840 = 10{,}000$ lbs.

$M = 12/12\ W\ l$

$= 12/12 \times 10{,}000 \times 12 = 120{,}000$ in. lb.

$M = R\ b\ d^2$ assume R = 100; b = 12,

$d^2 = M/R\ b$ $\qquad d = \frac{(M)^{\frac{1}{2}}}{(R\ b)^{\frac{1}{2}}}$

$d = \frac{(120{,}000)^{\frac{1}{2}}}{(100 \times 12)^{\frac{1}{2}}} = 10$

add 4" to this making d = 14".

Per cent of steel = 0.75 of 1%

Area steel in vertical slab at the foot,

A = .0075 x 14 x 12 = 1.26 sq. in. Use 7/8" ø rods.

Use the steel as follows:-

6 - 7/8" ø spaced 6" o.c.

4 - 7/8" ø " 8" o.c.

4 - 7/8" ø " 10" o.c.

Rest required " 12" o.c.

bond stress, $u = \frac{V}{\Sigma o\, j\, d}$

$= \frac{5000}{2 \times 2.749 \times .87 \times 14} = 75$ lb. per sq.in.

shear $= \frac{V}{b\, d}$

$= \frac{5000}{14 \times 12} = 30$ lb. per sq. in.

Design of Footing on the right:-

The footing on the right will be designed also as a continuous beam to take the pressure of the net vertical loads acting on it.

To simplify the calculation, the pressure will be assumed to cut the base at its center point, making the pressure of the base uniform.

Upward pressure on base = $\frac{25,150}{13.16}$ = 1850 lbs.

Downward pressure on base = 21 x 120 = 2510#.

Net difference = 2510 - 1850 = 660 # per sq.ft.

$M = \frac{12}{12} W l = 660 \times 12 \times 12 = 95,000$ in.lb.

$d = \frac{(M)^{\frac{1}{2}}}{(R\ b)^{\frac{1}{2}}} = \frac{(95,000)^{\frac{1}{2}}}{(100 \times 12)^{\frac{1}{2}}} = 8.9$ in.

make this 12 in.

percentage steel = 0.75 of 1%

area steel required = .0075 x 12 x 12 = 1.08 sq.in.

use 7/8" ϕ rods spaced 6" c. to c. This can be increased as the vertical wall is approached.

Bond stress:-

$u = \frac{V}{\Sigma o\ j\ d} = \frac{660 \times (12-1.5)}{2 \times 2.749 \times .87 \times 12} = 121$ lb. per sq.in.

Spacing of stirrups:

$P = \frac{V}{d} \times S$, using 3/8" rods at a working stress of 18,000 lbs. per sq. in.

$$1104 \times 18{,}000 \times \frac{9.0}{6930} = S \qquad S = 3.5 \text{ say } 4"$$

shear, $= V = \frac{660 \times 10.5}{12 \times 12} = 45$ lbs. per sq. in.

(This is probably all right, although 5# in excess of working shear.)

Design of cantilever on the left:-

The left cantilever was designed exactly in the same way and the following results computed.

Maximum toe pressure =

p = 3450 lbs. sq. ft.

The cantilever will be designed to take a uniform pressure equal to this:

M about the front of the vertical wall=

518,000 in. lb.

d = 20.5 say 24 in.

A_s = 2.16 sq. in. per lineal foot.

Use 1" - ø - 4½" o.c.

shear stress = $v = \frac{V}{b\,d} = \frac{3450 \times 5}{12 \times 24} = 60$ lbs. per sq.in.

Additional steel to take care of the excess shear is equal to (60 - 40) = 20 lbs. per sq. in., must be provided. Use 3/8"- ∅ - 4" c.c. all along the front cantilever.

Area on this spacing = 0.33 sq. in. per ft.

" required $\frac{20 \times 12 \times 24}{18,000}$ = 0.32 sq. in. per ft.

bond stress = 75 lbs. per sq. in.

Design of Counterfort:-

The counterfort will be designed as a cantilever beam, to take the horizontal component of the earth pressure, the resisting arm of the bending moment being measured perpendicular to the sloping side of the counterfort.

12 x 8800 = 105,600 lbs. per counterfort.

This acts through a lever arm of 7.0 ft.

B.M. = 105,600 x 7.0 x 12 = 8,850,00 in.lbs.

Depth of counterfort acting as a beam at

this point = 7.2 ft. and substracting 0.5 ft. from this for a covering of concrete over the steel leaves d = 6.7 ft.

$M = A_s f_s j d$

$d_s = M/f_s j d$ $j = 0.87$ $d = 6.7 \times 12$;

$f_s = 18{,}000$

$d_s = 8{,}850{,}000/18{,}000 \times .87 \times 6.7 \times 12$

$= 7$ sq. in.

Use 4 - $1\frac{1}{2}$" $\emptyset$ bars at the lower $\frac{1}{4}$, 3 of them being extended $\frac{1}{2}$, 2- $\frac{3}{4}$, and 1 all the way up to the top of the wall.

The counterfort wall is 18" thick.

The horizontal and vertical reinforcing in the counterfort designed to take tension only, will be composed of rods, figured or selected as follows.

The bursting pressure of earth at bottom of vertical wall,

$= 21 \times 40 \times 10.5 = 8{,}800$ lbs.

Area steel per ft. required = $\frac{8800}{18,000}$ = 0.5 sq.in.

Use $\frac{1}{2}$ in. ϕ rods spaced 8 in. c/c. with 2 in. a row for the lower half of the wall of $\frac{1}{2}$ in. ϕ rods spaced 16 in. c.c. for the upper half.

2. Pipe Line.

In hydroelectric developments wherein the water for power must be carried some distance through a penstock, either of wood stave or steel, the cost of the project is materially increased. Moreover, such pipe lines often offer almost insurmountable difficulties from an engineering viewpoint as well as from the financial viewpoint. However, in most developments, and in our case in particular, it would be impossible to develop but a small per cent of the power available, if a pipe line were

dispensed with. It is therefore the duty of the engineer, in general, to design a pipe line the cost of which will be the lowest consistent with true economy and safety.

The pipe line under consideration will extend from the foot of Lake Ripogenus to Big Eddy, a distance of 12,500 feet. It will be designed for a maximum head of 260 feet, and an average flow of 1150 sec.-ft. Since, with a head and flow of such magnitude a wood stave pipe becomes unfeasible both from a constructive and financial viewpoint, even with the saving of head resulting -- the pipe will be built of steel, supported at proper intervals by concrete piers. The profile and details of the pipe line are shown on plate 5.

The most economical conduit, carrying energy in any form whatsoever, is that wherein the annual cost of energy wasted is equal

the interest on the capital outlay, plus a certain per cent for depreciation and maintenance(Kelvin's Law). With this law as a criterion, the design of the pipe was carried out as shown in detail in the following paragraphs.

From the nature of the topography along the line of the penstock, it was decided to divide the pipe line into four divisions of 3100, 3200, 3100, and 3100 feet respectively(see plate 5), and to design each division separately in accordance with the above principle. Various diameters were arbitrarily chosen for each length and the most economical size computed. Only the correct and final computations are shown in table 4.

For the first length of 3100 feet and a diameter of 13.5 feet, the calculations are

Table #4.

Pipe Computation.

Dia. ft.	Area sq. ft.	Vel.	Head lost.	t	Vol. cu.in.	Wt.ft.+10%		Cost per ft. Dollars	Total Cost Dollars	10.5% Dollars	Lost Energy Dollar
					L = 3100	H = 100					
13.50	143.0	8.1	5.98	3/8"	2290	650	715	35.70	110800	11610	11960
					L = 3200	H = 140					
12.75	128.0	9.0	7.64	½"	2880	818	900	45.00	144000	15100	15280
					L = 3100	H = 200					
12.25	118.0	8.4	9.71	11/16	3810	1080	1188	59.40	184000	19300	19400
					L = 3100	H = 260					
12.00	113.1	10.13	11.13	13/16	4420	1255	1380	69.00	214000	22500	22260
				Total cost of pipe line =					$ 652800		

given below:-

A = 143 sq. ft. Q = 1150 sec.-ft.

$h_f = 4LV^2/C^2D = 4x3100x(8.05)^2/(100)^2$ 13.5

= 5.98 ft.

t = 2.6 h d /S = 2.6x260x13.5/10,000

= 0.375 in. (approx.)

Volume of steel per foot,

= cir. x t = 508.9x0.375

= 2290 cu. in.

Weight per foot,

= vol. x w_s = 2290 x 0.284

= 650 lbs.

Adding 10% for details,

= 650 + 65 = 715 lbs.

The cost per pount for the pipe in place has been taken at 5 cts., an average price. The cost per foot will then be,

= 715 x 0.05 = $35.70

Total cost,

= 35.70 x 3100 = $110,800.

The life of the pipe has been assumed as 20 years, which gives a depreciation rate of 3%. The interest on the cost will be 6%, and the maintainence at 1.5%. The total annual charges of the pipe line will then be

$$0.105 \times 110,800 = \$11,610$$

In a hydroelectric plant where the power available is unlimited and the market limited, it is customary to value the wasted power at about the actual cost of generating; but where the available power is limited and the market practically unlimited, the selling price at the sub-station is adopted as the cost of wasted energy. In our case the latter supposition has been assumed, and the selling price decided as $20 per horsepower-year.

Since the average power sold is 26,000 hp., and the head available to the pipe line is 260 ft.,the value of one foot of head is ,

$$(26,000/260)20 = \$2000.$$

The total cost of energy wasted by friction in the pipe then is,

h_f x 2000 = 8.05 x 2000 = $11,900 yr.

Annual charges = $11,610 yr.

It is thus seen that the 13.5 foot pipe fulfills the law in that the cost of wasted energy equals the annual charges on the pipe, and this diameter was accordingly chosen. The calculations for the other lengths of penstock with the computed diameters are shown in the table. It will be noticed that the pipe decreases from 13.5 ft. at the intake to 12.00 ft. at the feeders.

3. Scroll Cases.

In order to secure the highest possible efficiency for a vertical waterwheel, the water must impinge upon the runners at all points

about the circumference with equal force and velocity. In order to secure this result, the water issuing from the feeders is led around and into the water wheel through a "scroll case". This is a concrete conduit of scroll outline, whose area decreases from the feeder diameter, in such a way as it passes around the wheel, that the volume and velocity of the water entering the wheel remains a constant quantity. The scroll cases of all units have been so designed; the actual procedure being almost identical with that used in designing the scroll cases at Keokuk, the engineer in charge of which was and is one of the foremost men in this particular line in the country.

Design:-

From hydraulics:

$$H.P. = QhE \times 62.5/550$$

$$Q = 550 \times H.P./h\ E \times 62.5$$

$= 550 \times 10{,}000/255 \times .90 \times 62.5$

$= 383$ sec.-ft.

The diameters of the feeders to the units are 6.0 ft. Engineering practice has shown that the velocity of water entering scroll cases is equal to $0.12(2gh)^{\frac{1}{2}}$,-

$$V = 0.12\ (2 \times 32.2 \times 255)^{\frac{1}{2}}$$

$= 15.4$ ft. per sec.

The velocity of the water in the feeders:-

$$V = 383/(\pi D^2/4)$$

$= 13.3$ ft. per sec.

The area of the scroll case at the entrance must therefore be such as to change the velocity from 13.3 to 15.4 ft. per sec.; and each succeeding section must be decreased, as the volume of water left decreases, so as to keep this velocity constant. The scroll case has therefore been divided up into twenty sections, whose areas vary almost directly as the amount of water therein.

Then each section will pass into the wheel one twentieth of 383 = sec.-ft. or 19.15 sec.-ft. Therefore the first section will carry 383 sec.-ft., the second section 19/20 of 383 or 364 sec. ft., and so on till the last section, which carries 1/20 x 383 or 19.15 sec. ft. Now, since $Q = V A$, or $A = Q/A$, we can get the area of each section by dividing the respective volumes by the constant velocity 15.4 ft. per sec. However, in the smaller areas, the hydraulic radius is less and the resistance to flow correspondingly greater. Hence, to the area of each of the sections after the first, there is added a certain per cent, varying from 1½% in the second section to 28½% in the last. The shape and position of each of the sections is shown in plate 6, and the computed results are shown in table 5.

Table 5.

No.	Area.	+ %	=	+	=	ht.	wdth.
1	24.85	0.0	24.85	1.96	26.81	6.0	4.46
2	23.62	1.5	23.95	1.96	25.91	6.0	4.46
3	22.38	3.0	23.04	1.96	25.00	6.0	4.16
4	21.12	4.5	22.8	1.96	24.04	6.0	4.01
5	19.88	6.0	21.8	1.96	23.04	6.0	3.84
6	18.62	7.5	21.10	1.96	22.06	6.0	3.68
7	17.40	9.0	19.00	1.96	20.96	6.0	3.50
8	16.17	10.5	17.86	1.96	19.72	6.0	3.30
9	14.92	12.0	16.71	1.96	18.67	6.0	3.11
10	13.68	13.5	15.52	1.96	17.48	6.0	2.91
11	12.42	15.0	14.20	1.96	16.24	6.0	2.71
12	11.18	16.5	13.04	1.69	14.73	5.8	2.58
13	9.94	18.0	11.72	1.44	13.16	5.6	2.37
14	8.70	19.5	10.4	1.21	11.61	5.4	2.18
15	7.46	21.0	9.03	1.0	10.03	5.2	2.0
16	6.21	22.5	7.61	0.64	8.25	4.8	1.74
17	4.94	24.0	6.12	0.36	6.48	4.4	1.49
18	3.73	25.5	4.68	0.16	4.84	4.0	1.21
19	2.49	27.0	2.16		3.16	3.6	0.93
20	1.24	28.5	1.60		1.6	3.2	0.50

4. Draft Tubes.

In order to take advantage of the total head available to a vertical water wheel, it would be necessary to set the wheel below the surface of the tail race water level. This setting, however, would produce a loss of efficiency in the wheel itself. It also produces certain complications in the setting of the wheel. To overcome these difficulties; to keep up the efficiency of the wheel, and at the same time to retain advantage of the total head, the hydraulic engineer has evolved the"draft tube". In this type of installation the turbine is set above the tail race level (not more than 34 feet theoretically). Leading the water away from the turbine is a concrete pipe, the cross-sectional area of which is gradually enlarged so that the velocity of the contained water is reduced, from the high value at which it leaves the turbine runners, to a value of

from two to five feet per second, on entering the tail race. The top of the draft tube must always be a foot or more below the lowest known level of the water in the tail race. Moreover, there must be sufficient depth below the bottom of the draft tube where it enters the tail race, so as to allow the escaping water to flow away with a minimum amount of friction. It can be easily seen that with a draft tube so designed, there is created in the latter a vacuum, which produces a sort of a forced draft in the pipe, and so increases the output of the turbine. The outline of the draft tube is shown in plate #4.

The center line of the tube is a quarter circle, with short tangents at the top and bottom. The resulting outline changes the direction of the water gradually so as not to produce eddy currents in the pipe,- from the vertical direction to that of horizontal. The sections of the draft tube are circles at the top and at the bottom,

rectangles with semi-circles at the ends, with the longest dimension parallel to the surface of the tail race,- the change being made to reduce the necessary excavation under the power house and in the tail race.

The outlet of the turbine is $5\frac{3}{4}$ ft. in diameter. Dividing the area of this section into the volume of the average full load discharge gives the velocity at the entrance of the draft tube:

$$V = 383/0.7854 \times 5.75^2$$

$$= 14.75 \text{ ft. per sec.}$$

Since the outlet velocity was chosen at 3 ft. per sec., a change of 14.75 - 3.0 = 11.75 ft. per sec. must be made. The length of the draft tube is accordingly divided up into twenty-five sections, the areas of which are increased in such a ratio as to produce a constant retardation of 0.47 ft. per sec. per section. In table #6, column 2 gives the velocities, decreas-

Table #6.

No.	Vel.	Area	Dia.	C-C
1	14.75	26.00	5.75	
2	14.28	26.82	5.85	
3	13.81	27.74	5.94	
4	13.34	28.70	6.05	
5	12.87	29.87	6.16	
6	12.40	30.90	6.27	
7	11.93	32.08	6.39	
8	11.46	33.42	6.52	
9	10.99	34.82	6.66	
10	10.52	36.40	6.81	
11	10.05	38.10	6.97	
12	9.58	39.90	7.25	
13	9.11	42.02	"	0.01
14	8.64	44.30	"	0.42
15	8.17	46.90	"	0.77
16	7.70	49.75	"	1.17
17.	7.23	53.00	"	1.62

Table #6.(Con.)

No.	Vel.	Area	Dia.	C-C
18	6.76	56.70	7.25	2.13
19	6.29	60.90	"	2.71
20	5.82	65.90	"	3.39
21	5.35	71.50	"	4.17
22	4.88	78.40	"	5.12
23	4.41	86.90	"	6.30
24	3.94	97.20	"	7.70
25	3.47	110.30	"	9.51
Exit	3.00	128.80	"	11.95

ing by 0.47 for each section. Dividing these velocities into the constant quantity 383 sec. ft. gives the respective areas,- column 3. In column 4 the diameters of the circular sections are given. It will be noticed that begining with section #13 the section is changed to the rectangle with semi-circles at the ends. The height of the rectangle, or the diameter of the above semi-circles is kept constant, while the distance between centers is increased to produce the desired increase in area.

To produce a steady and unhampered flow of water from the draft tubes, and to afford protection from logs and the resulting debris, during the log drives, it is proposed to construct a concrete retaining wall about 50 or 60 feet long, from the edge of the power house out into the tail race(Big Eddy), parallel to the natural course of the river.

PROPOSED HYDROELECTRIC DEVELOPMENT ON THE WEST BRANCH OF THE PENOBSCOT RIVER, MAINE.

Part 111.

POWER HOUSE.

A. General Plan.

The power house, located at the edge of Big Eddy, will be a three story, reinforced concrete structure, its overall dimensions being 63 ft. wide by 211 ft. long by 100 ft. high, measured from the centre of the draft tube exit. The exterior will be of monolithic appearance, rising straight from the waters edge, devoid of any coping or architectural features, except for the necessary weather coping at the extreme top. The surface finish will have a "stucco" appearance.

On the river side of the building, lighting will be effected by 7 steel-sash windows, 20 ft. wide by 34 ft. high. This type of window, while being of modern and fireproof design will add rather than detract from the appearance of the structure. On the land side, all

windows have a standard width of 14 feet. Entrance to the building will be at the north end, through the general office.

On the first floor will be the generator room, the transformer compartments, and auxillary machine hall. At one end of the building are located the shop and offices. A railroad siding will enter the shop.

The second and third floors will be supported by I beam columns and girders, with interconnecting concrete beams.

Below each unit will be a room containing: governor pumps and shafting; separate triples oil pumps driven from the governor shafting, to supply oil under pressure for the thrust bearings of the units; and the operating mechanism for the "Butterfly" valves

of the individual feeders. These rooms will be connected by a tunnel, to which access may be had from the generator room.

Finally, below the machine hall, there will be a second tunnel, connected to the first at suitable intervals, which will contain the main transformer oil tanks, and their filtering and drying apparatus. Since the presence of one-half of one per cent of water in transformer oil drcreases its insulating qualities fifty per cent, the need of adopting such measures and having such appartus on hand as will keep this oil absolutely free from moisture is apparent.

The second floor, at the shop end, occupies the full width of the building, and contains the feeder switches and lightning

arresters. For the full length of the generator room, the second floor is a gallery 20 feet wide, extending over the machine hall to the edge of the main transformers; and contains the high tension switches, and in the center the main control board.

The third floor is similar in construction and holds only the high tension bus bars, which extend the whole length of the building. The line reactance coils, and feeder entrances are at the shop end of the gallery.

B. Main Equipment

1. General.

Before any step can be taken toward the choice of power-house equipment, a number

of facts must be definitely determined beforehand.

First the load and load-factor, both for the present and future must be known, as this effects the number and size of units which can be used. The choice of frequency is also a fact which may give some trouble. This has been more true in the past than at the present time. Now, the use of 60 cycle current is almost universal both for industrial and lighting loads. Three phase generation has always been used in hydroelectric stations of large power and distant transmission.

In order to analyze the demand for power, it is necessary to draw a load curve for the probable demand for power. The load curve of a generating station shows the demand for power per hour for a 24 hour day, the hours being plotted as abscissae and the power as ordinates.

If the power is already contracted for, this is an easy matter. The latter assumption has been made in our case.

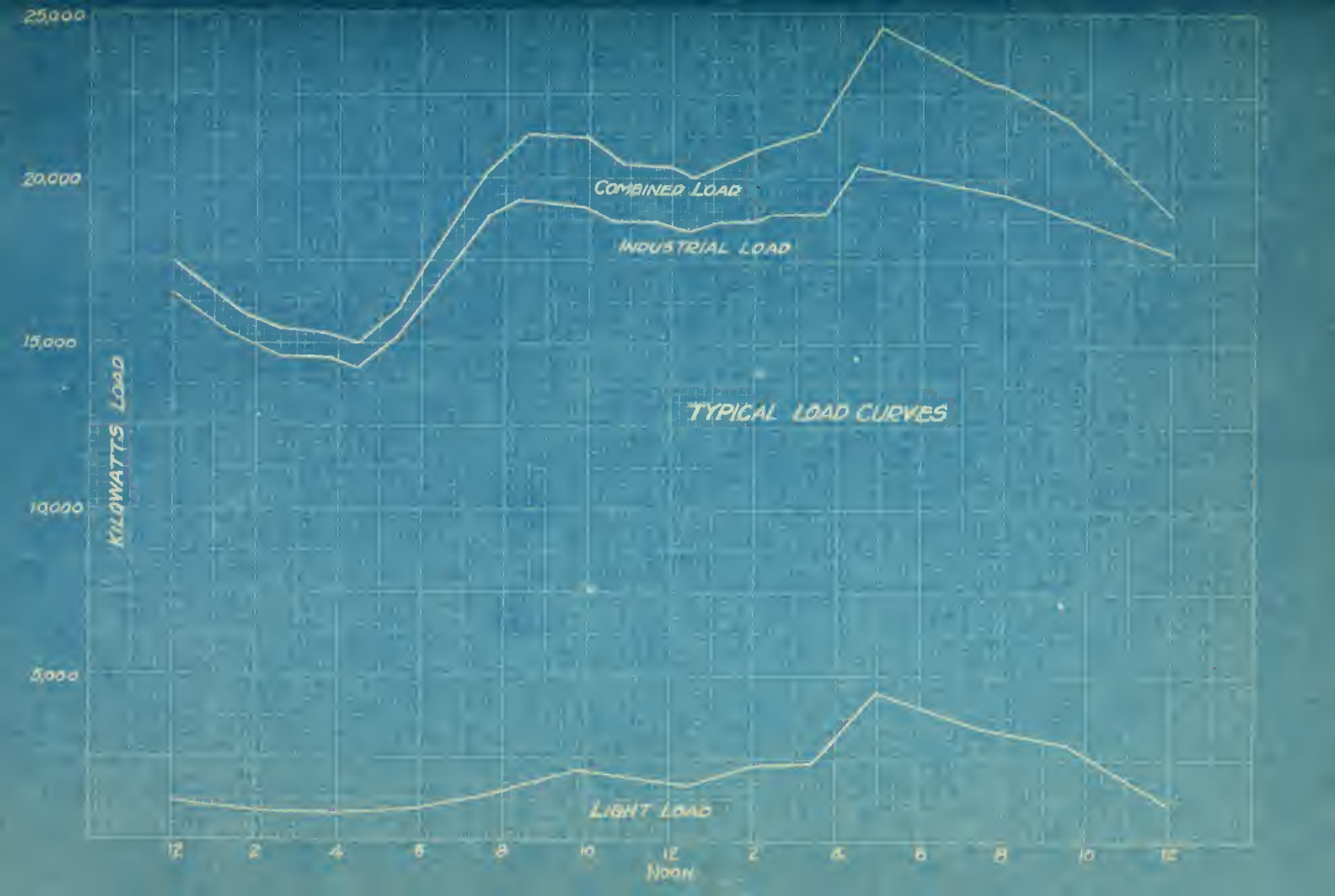
Because of the increasing use of three eight hour shifts by factories, especially in New England, a 24 hour industrial load curve was chosen. To this was added a lighting load equal to 20% of the total peak load.

In order to find the average load and the power factor, the area beneath the curve was measured, and divided by the total abscissa which gave the average power. Then dividing the average power by the peak load gave the load factor:-

average kw./peak kw.	= load factor
19,000/24,500	= 0.8
19,600/0.746	= 26,300 hp.

Assuming the following efficiencies:-

Generators	= 95%
Transformers	= 99%
Turbines	= 88%


25,000
20,000
15,000
10,000
5,000
KILOWATTS LOAD
COMBINED LOAD
INDUSTRIAL LOAD
TYPICAL LOAD CURVES
LIGHT LOAD
12
2
4
6
8
10
12
Noon
2
4
6
8
10
12

The average gross hydraulic horsepower will be,

$$hp. = 26,300/.99 \times .95 \times .88$$

$$= 31,400$$

$$hp_{(max.)} = 31,400/0.8$$

$$= 39,200$$

In order to carry this load, it was decided to install five 10,000 hp. turbines, connected to five 6750 kv-a. Generators, one unit being for spare, and to carry any extra overload. The output of the four generators at o.8 power factor is,

$$kw. = 4 \times 5,400 = 21,600$$

$$= 21,000/0.746$$

$$= 28,400 \text{ hp.}$$

Carrying 25% overload at 0.8 power factor,

$$hp = 35,400$$

Since the average and maximum load demands are respectively, 26,300 lbs., and 32,800 hp., it is seen that four generators will be working

at full load, and highest efficiency most of the time, and can also take care of the peak load, if it is not desirable to throw in the fifth unit.

The turbines were chosen about 30% greater capacity than the generators in order to allow the latter to carry an overload, and to allow for a decrease in efficiency of the turbines which would occur in time.

2. Hydraulic.

The five 10,000 horsepower vertical shaft Francis turbines; and the single 500 horsepower, exciter turbine, will be of the G. F. type furnished by the Allis Chalmers Manufacturing Co. They will operate normally under a head of 250 feet, but will be designed to operate efficiently under a varying head of from 240 to 260 feet. A

maximum efficiency of between 90% would be expected of the above type of wheel.

The entire weight of the turbine and generator will be supported from a special thrust bearing at the top of the unit, to which oil will be forced under probably a 200 pound pressure from the pumps below the unit.

Lubricating oil will be pumped from the tunnel beneath the machine hall to each unit.

The regulation of each turbine will be accomplishes by a specially designed Allis Chalmers oil pressure governor, the balanced guide vanes being controlled through an exposed operating mechanism from the actuators, which are located on the generator floor in front of each unit. The operating mechanism of such a governor consists of a rocker ring, which is carried on ball bearings and which is

connected by links to the cranks on the guide vane stems. The rocker ring is operated by means of piston rods from two high pressure regulating cylinders, which together with the relay valves, and the induction motor-driven triplex pumps to furnish oil for the cylinders mentioned above, are located in the rooms under the respective units. As is standard with such governors, a steady speed within one-half of one per cent, would be expected to be maintained; and on decrease of load to bring the speed to normal within five seconds.

3. Electrical.

The five main generators will be furnished by the General Electric Co. and will be of the three-phase, sixty cycle, 6600 volt, revolving field type, with a normal rating of 6750 kv-a.

An efficiency of about 95% would be expected at full load and 0.8 power factor. According to standard practice the generators would be expected to carry an overload of 25% for two hours; a 50% overload for one hour, and finally a 100% overload for two minutes without damage.

Excitation of the generators will be furnished by two induction motor-driven, 375 kw. 250 volt, direct current generators; and one 500 horsepower turbine-driven generator of the same type. Any two of these generators are sufficient to carry the complete excitation load. The excitation group is located in the approximate center of the power house.

The main transformers, five in number, are three-phase water cooled and oil insulated, of the shell type, designed to raise the voltage from 6600 to 90,00 volts. Their normal rating

will be 6750 kv-a. An efficiency of 98.5 to 99.0% would be expected. The design will be such that the cover, core, and leads can be lifted out of the tanks without detaching any of the parts, while the whole unit rests on a wheeled truck which permits moving of the transformer from its separate compartment out into the generator room, where it may be handled by the crane. The water for cooling purposes will be pumped from the tail race by induction motor-driven centrifugal pumps, after which it is filtered and led to the cooling coils. This operation being performed in the tunnel beneath the machine hall.

Some doubt was at first expressed as to whether single or three phase transformers should be used. After some discussion, three phase transformers were decided on for the following reasons:-

a. Increase in efficiency.

b. Decrease in cost.

c. Better for transportation.

d. More compact and smaller power house.

Moreover, since many large power companies, such as the Commonwealth Edison, Mississippi Water Power Co., and many others, have used three phase transformers for some time with entire satisfaction, the evidence in favor of three phase over single phase transformers seemed sufficient for the choice of the former.

There will also be four auxiliary transformers, for stepping down the voltage from 6600 to 440 volts. These will be single phase, shell type, water cooled, machines, with a normal rating of 350 kw. The purpose of these transformers is to supply power for the 440 volt induction motors driving the exciters; and for all the other auxiliary power used in the plant.

Two compensators of the standard type are provided for starting the induction motors.

The main switches are between the transformers, and high tension bus. These are of the General Electric H_6 motor operated type, and are operated from the control board. They are also equipped with overload relay so that if the current reaches 60% overload the switch opens automatically. A K_2 non automatic oil switch is placed between the generator, and the transformer , and between the generator and transfer bus. These are operated from the control board. The feeders are provided with 90,000 volt H_6 switches arranged for automatic cut out on overloads.

4. Control Board.

The entire plant is controlled from a seven panel desk type control board, located

on the switch floor opposite the exciters at the center of the powerhouse. The instruments are on a board placed over the front of the desk so that they may easily be read by the operator while operating the switches.

Each generator panel is equipped with the following apparatus. Control switch for 90,000 volt H_6 overload automatic switch between station bus and machine transformer. Control switch for K_2 non automatic switch at generator. Control switch for K_2 non-automatic oil switch connecting generator leads to transfer bus. Control switch throwing the alternator field on the exciter mains. These four control switches are all provided with red and green lights so that the operator will know if the switches are in or out. There are also control switches for a series motor

to regulate the field rheostat so that it may be operated from board and a control switch for series motor to regulate turbine governor when bringing the alternator into synchronism and in distributing the load.

A red signal lamp is provided to light when machine is overloaded. This is connected to a overload relay, which closes the lamp circuit upon overload. A voltmeter and synchronizing receptacle are provided and a ground detector plug. Above on the instrument board, we have an ammeter for each phase, a field ammeter, a power factor meter, and a Polyphase indicating wattmeter. Back of the desk board we have a polyphase recording watthourmeter. At the right hand end we have a swinging panel containing the a.c. voltmeter, and frequency meter and the Lincoln synchroscope.

The sixth panel controls two induction

motor-driven exciters. These are provided with a switch which controls a double pole circuit breaker that connects the exciter on the line, a switch to control a single pole circuit breaker in equalizer lead. A control switch which controls the compensator which starts the motor. These three are provided with red and green lights to show if circuits are closed or open. There is also a switch to regulate the resistance in field circuit. The board is also provided with voltmeter plug and ground detector. Each machine has its ammeter. The last panel controls the third exciter. It is equipped the same as the other two, except that it has no compensator switch, but has a switch to control the turbine governor. This panel has the switches for the two outgoing feeders. A d.c. voltmeter is provided at this end on the instrument board.

PROPOSED HYDROELECTRIC DEVELOPMENT ON THE WEST BRANCH OF THE PENOBSCOT RIVER, MAINE

Part IV

TRANSMISSION LINE.

A. Location.

A transmission line has been designed to transmit the total power to Bangor, approximately 90 miles distant. It is believed that a fair percentage of the power could also be sold en route to the numerous towns and cities, with their paper and lumber mills. However, the greater part of the power would be sold in Bangor, which exceeds all other surrounding cities in population and industries.

The route followed would probably be along a branch of the Bangor and Aroostook Railroad, which passes along the shore lines of Lake Chesuncook on the south, to its junction with the main line of that road, and thence along this route to Bangor.

General description of line:-

Two three-phase circuits of No. 0 B. & S. stranded copper cable will be run in parallel on one set of steel towers, spaced approximately

600 feet apart. The towers will be of the semi-flexible type, with rigid strain towers at intervals of about a mile, or more frequently where corners or extra long spans render their use necessary. Two 3/8" galvanized Siemens-Martin steel-strand cable will be carried the full length of the line and be firmly secured to the top of each tower.

B. Electrical.

One of the first questions which arises in the design of a long distance, large-power transmission line, is that of the economic voltage to be used. A rough rule calls for a thousand volts per mile of transmission. However, in our case, a preliminary choice of 90,000 volts was based on a formula advocated by Alfred Still in his "Overhead Electric Power Transmission",which is given below with the specific computations of

our case.

$$V_k = 5.5 \text{ (distance + hp./200)}^{\frac{1}{2}}$$
$$= 5.5 \text{ (90 + 33,000/200)}^{\frac{1}{2}}$$
$$= 87,500 \text{ kilovolts.}$$

The final value of 90,000 volts was decided by applying "Kelvin's Law" on the economical transmission of power, the application of which was explained in detail in the design of the penstock(see page)

Various sizes of wires were arbitarily chosen and for each wire, two or three values of voltage were assigned and the cost of transmitting the energy over each of these wires at each of the chosen voltages was estimated. Against the cost of these sizes and votages[1] was balanced the probable selling price of the energy lost by the resistance to the current in each of the wires at the different voltages chosen. In the cost of each of the

lines was included the cost of towers , conductors, insulation and erection. That size of conductor and that voltage of transmission wherein the cost of wasted energy was equal to the yearly charges on the line was chosen as the most economical size of conductor at that voltage.

Cost of 85 miles of conductor, 6 wires on line at 20¢ per lb.

Size	Cost	Int. at 8%
4/0,	$352,000	$28,200
3/0	278,000	22,200
2/0	220,500	17,650
0	173,800	13,900
1	137,200	10,970

Cost of pole line, complete without wires, from table in Still Pp. 39

Voltage	Cost	Per Year
80,000	221,000	19,900
90,000	240,000	21,600
100,000	264,000	23,800
110,000	293,000	26,400

Maximum power to be transmitted is 24,500 kw. Load factor is .8. Power factor is .8.

Current per wire, using double system is

80,000 = 110.5 amps.

90,000 = 98.0 "

100,000 = 88.5 "

Kilowatts lost in line for various sizes of wires and votages.

	80,000	90,000	100,000
4/0	1,040	814 .5	665
3/0	1,302	1,020	834
2/0	1,640	1,282	1048
0	2,075	1,621	1320

Cost of power lost at $7.00 per hp. year.

	80,000	90,000	100,000
4/0	$ 9,720	$7,600	$ 6,215
3/0	12,200	9,540	7,790
2/0	15,315	12,000	9.790
0	19,390	15,185	12,385

Cost of 80,000 Volt line per year.

	2/0 Wire	0 Wire
Pole line	$19,900	$19,900
Power lost	15,315	19,390
Wire	17,650	13,900
	$ 52,865	53,290

Cost of 90,000 volt line per year

	2	2/0	0
Pole Line		$21,600	$ 21,600
Power Lost		12,000	15,180
Wire		17,650	13,900
		$51,250	50,685

Cost of 100,00 volt line per year

	0	1
Pole Line	$23,800	$ 23,800
Power Lost	12,385	15,600
Wire	13,900	10,970
	$ 50,085	$ 50,430

There is very little difference in cost per year of line between 90 and 100, so that the additional cost of 100 kv power house puts this out of the question.

Line Calculations:-

Star Voltage = $90,000/(3)^{\frac{1}{2}}$ = 51,900 volts.

Distance between wires = 122, 122 and 244 in.

Effective distance = $122 \times 2^{1/3}$ = 153.8 in.

The voltage induced per mile =

$$.004656 \times f \times I \times \log (2.568\ D/d) \times 90$$

$$= 7480 \text{ volts}$$

The resistance drop = $98.2 \times 90 \times .518$

$$= 4575 \text{ volts}$$

Capacity = .0388/ log D/r = .01329 m.f. per mi.

The Capacity current

$$I_c = 2\pi f \times (.01329/2) \times 90 \times 51.9$$

$$= 11.69 \text{ amps.}$$

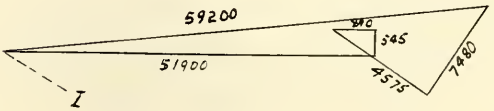
The resistance drop, due to capacity current

$$V_{cr} = 11.69 \times 90 \times .518 = 545 \text{ volts}$$

Inductance Volts, due to capacity current

$$V_{ci} = .004656 \times 60 \times 11.69 \times 3.0285 \times 90$$

$$= 890 \text{ volts}$$



Voltage at generator = 59,200 volts

At no load with generator voltage at 59,200,

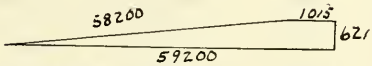
I_c = 2 π f x .01329/2 x 90 x 59,200

= 13.32 amps.

V_{cr} = 13.32 x 90 x .518 = 621 volts

V_{ic} = .004656 x 60 x 13.32 x 90 x 3.0256

= 1,015 volts



Regulation = (58200 - 51,900)/ 51,900

= 12.13%

C. Mechanical.

Economical length of Span:-

The loading on wires is taken at one-half inch of sleet and a wind velocity of fifty miles per hour.

W = w + 1.254 x r(d + r) =.840 lbs. per ft. wire

The wind velocity gives a load of

dV^2 / 5,000 = .685 lbs. per ft.

Total load $= (.84^2 + .685^2)^{\frac{1}{2}} = 1.084$ lbs. per ft.

The Area of #0 wire is .083 sq. in.

The maximum tension allowed at 28,000 lbs. per sq. in. = 2.324 lbs.

The tension in wire with a sag of s ft.

$$P_{max} = Wl^2/8\ s + Ws$$

The sag for the different spans to keep the maximum stress at the above figure is:-

Span in Ft.	Sag	Span	Sag
500	14.6	700	28.7
550	17.6	750	32.8
600	21.1	800	37.3
650	24.7	850	42.1

The economical length of span was computed by the following method:-

Table #6.

Span	Sag	Ht. Tower	Total Cost	Cost per ft. of line
550	17.6	65'-0"	134.70	0.245
600	21.1	68'-10"	146.20	0.240
650	24.7	72'-11"	159.80	0.246

It is thus seen that the 600 ft. span is

the most economical. The spacing between the conductors is 122 inches.

Towers.

Two types of towers are used as has been stated,- the flexible type and the rigid-strain tower. Every tenth tower is of the rigid type.

Flexible towers are designed to take full weight of the wires, sleet, wind etc., and the bending moment caused by the wind blowing across the line at 50 miles per hour. The rigid towers are designed to take a breast pull that would be present if all the wires on one side should be severe. In this case, two feet would be added to the effective length of the wire, increasing the sag. The length of the wire with maximum loading and with a sag of 21.1 feet is equal to

$$l + 8s^2/3 \times l = 600 + 8 \times 21.1^2/3 \times 600$$

$$= 601.98 \text{ ft.}$$

If all the wires on one side break, we must add two feet to this length, which will give 604 feet.

$604 = 600 + 8 \times s/3 \times 600$

$s = 30$ ft. in wires on other side.

$P = W l^2 / 8 \times s = 1626$ lbs. in each wire.

For the steel ground wires, the stress is 1838 lbs.

The towers are designed as shown on plate 14. The stresses are figured for each member in both the flexible and rigid strain towers. The stresses from the dead load, the wind load, and the breast pull are figured separately, and the maximum stress that can be applied on any one member is determined.

The flexible towers are designed for use on straight runs only; but if the deviation from the straight line does not exceed five degrees, these intermediate supports may be equipped with

strain insulators, and used at corners . For angles greater than five degrees, but not exceeding a limit of eight degrees, these supports may be used with two guy wires to take the transverse stress due to the resultant pull of the wires. If the approach spans are reduced to 340 feet, an eight degree curve may be turned on a semi-flexible structure without guy wires.

Strain towers shall be used for turning corners up to 30 degrees; but when the total deviation exceeds this amount, two towers must be used.

All the members of both types of towers are of angle steel rivited to gusset plates at all connections. The tension members are designed to take a maximum stress of 20,000 pounds on the net section. The maximum stress

allowable on compression members will be given by the following formula:-

$$S = 20,000 - 100 \times l/r$$

The ratio of l/r is limited to 120 for main members, and 150 for struts.

The minimum thickness of metal for main uprights and crossarms is one-fourth of an inch, and for other members three-sixteenths inches. In the rigid tower, the stress in the main uprights in the bottom of the section is 1,525 pounds, due to the weight, 23,400 pounds due to breast pull, and 8,230 pounds due to the wind. This gives the total of 32,155 pounds.

The unsupported length of the members is ninety-six inches. Therefore since l/r = 120, r = 0.80. An angle 5"x 5"x 3/8"seems to be large enough. The minimum r is 0.99; the area is 3.61 sq. inches; Therefore, the strength is

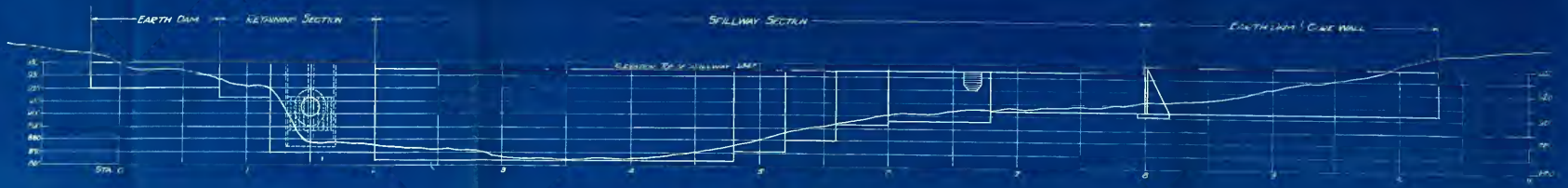
$$S = 3.61(20,000 - 9,600/99) = 37,183 \text{ lbs.}$$

The strength of a $2\frac{1}{2}$" x2"x3/16" angle used as a minimum tension member is

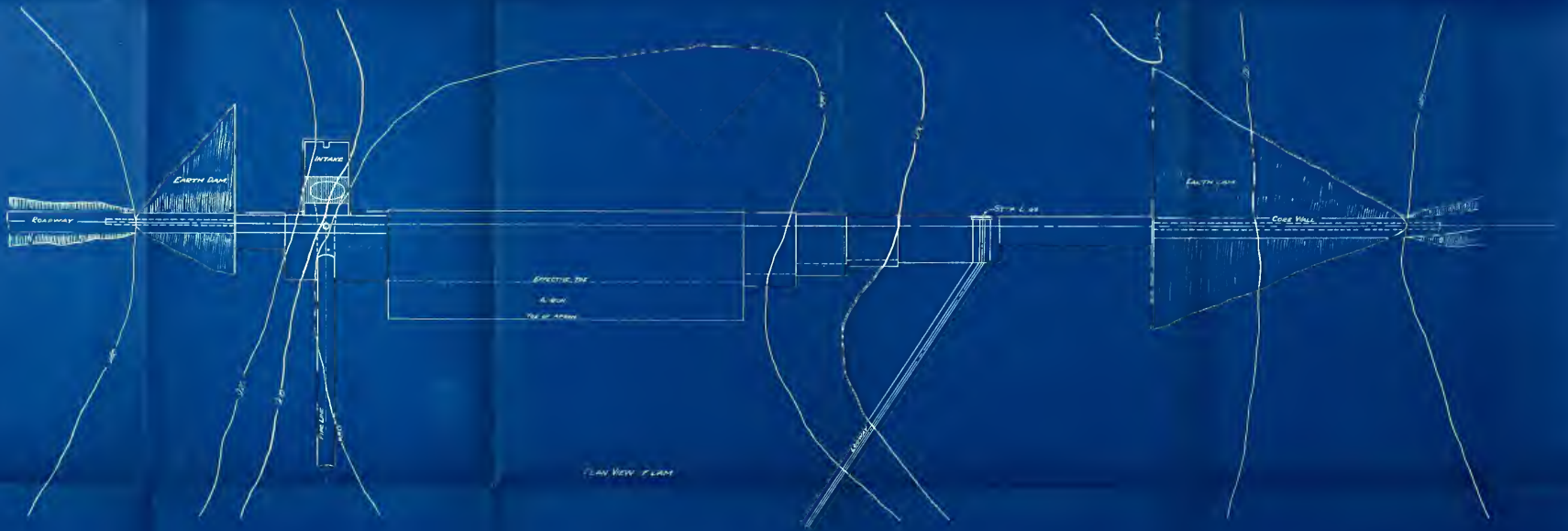
20,000(0.81 - 0.14) = 13,400 lbs.

This is greater than any of the tension stresses, and this size will be used throughout.

The size of each of the members of both the towers is shown on plate 14.



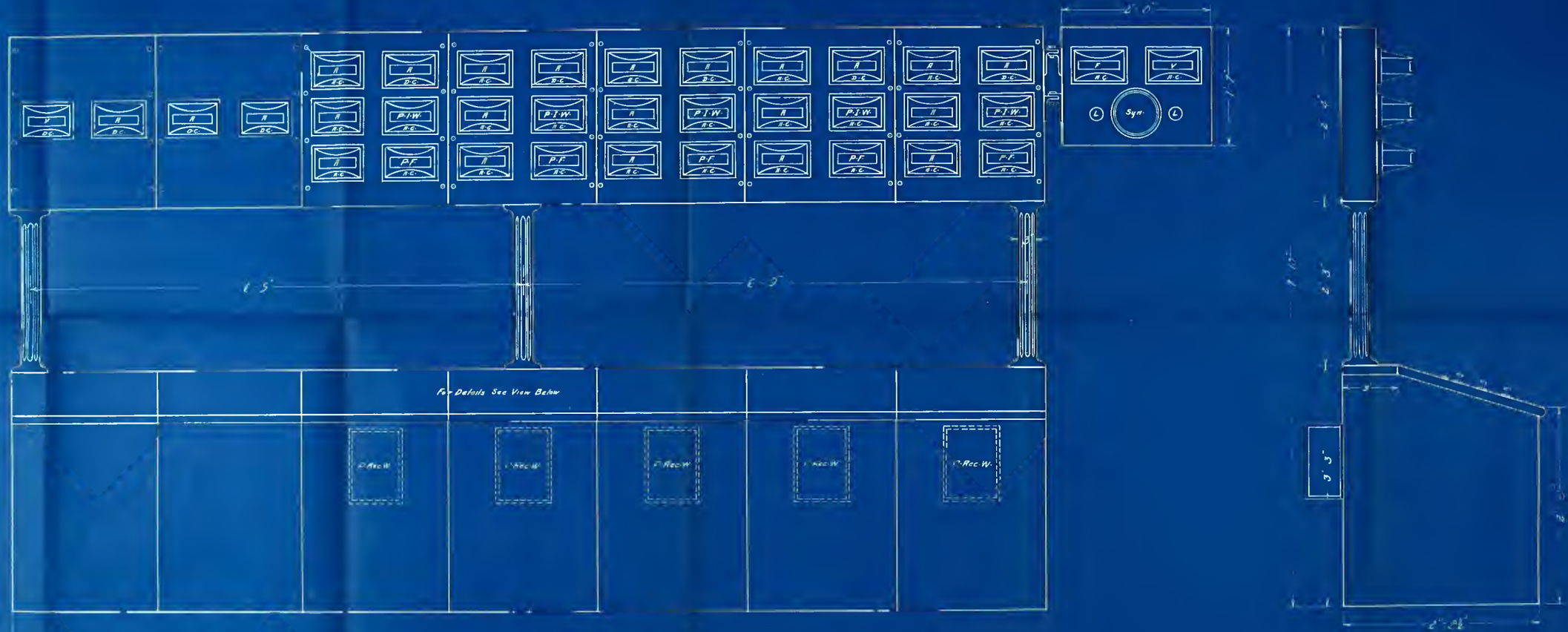
PROFILE AT DAM SITE AND SECTIONS OF DAM



PLAN VIEW OF DAM

PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.
DAM LAYOUT AND TOPOGRAPHY

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 1930 PLATE 2 SCALE 1"=800'



PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

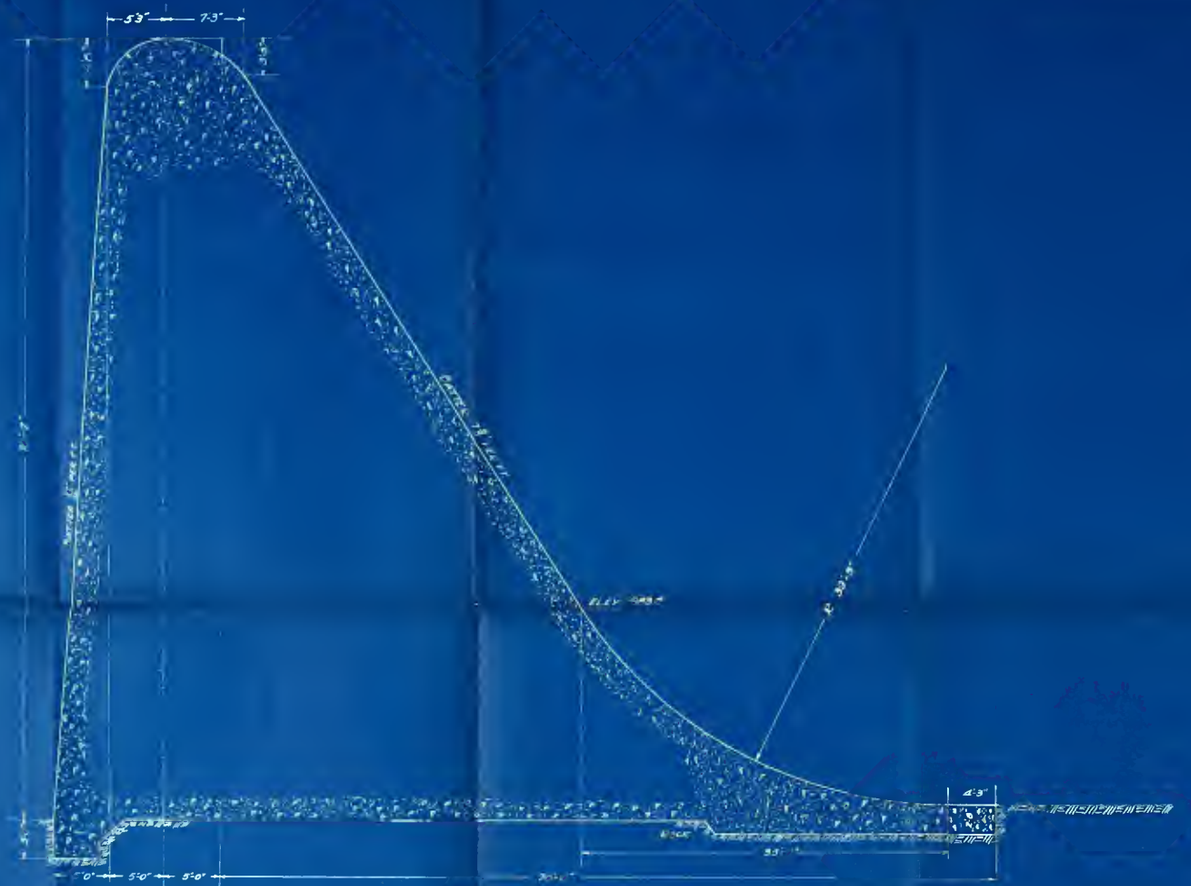
CONTROL BOARD

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1917 PLATE-13 SCALE-1/2"=1'-0"
J.R. CLINE, INSPECTION ENGINEER, WATERVILLE

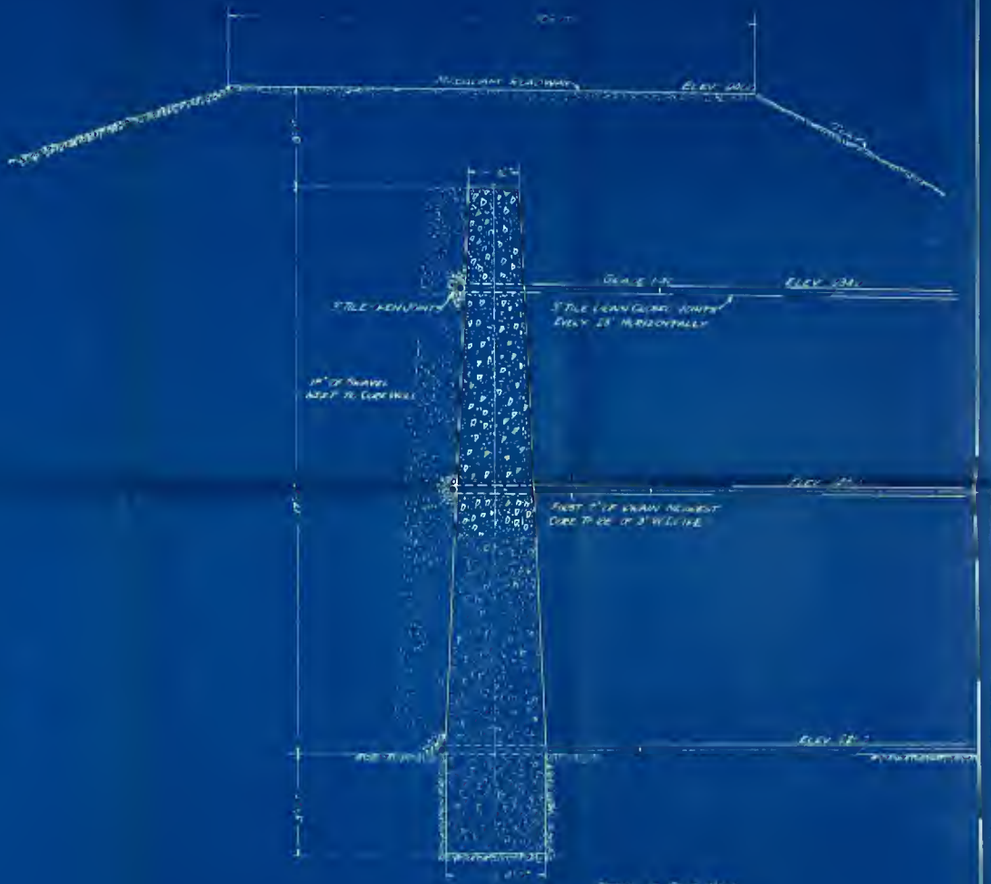
KATAHDIN HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

MAP OF STORAGE BASIN

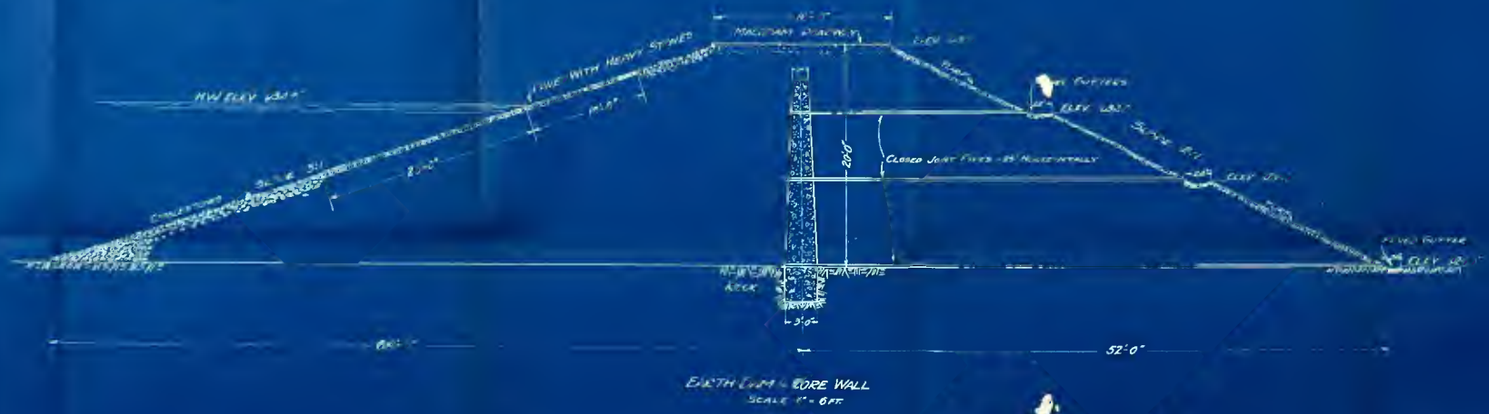
MAINE INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1938 PLATE 1 SCALE 1/2000
J. STOLLNACK HOUSTON-ROSE EISENHAUER



SPILLWAY SECTION
SCALE 1"=8FT.


ELEV. 381'
STILE ADJUNCTS
STILE LEAN CLOSE JOINTS EVERY 12' HORIZONTALLY
12" OF GRAVEL NEXT TO CORE WALL
ELEV. 374'
FIRST 2" OF CRUSH NEXT TO CORE TO BE 1/2" WELDING
ELEV. 12'

DETAIL OF CORE WALL
SCALE 1"=8FT.

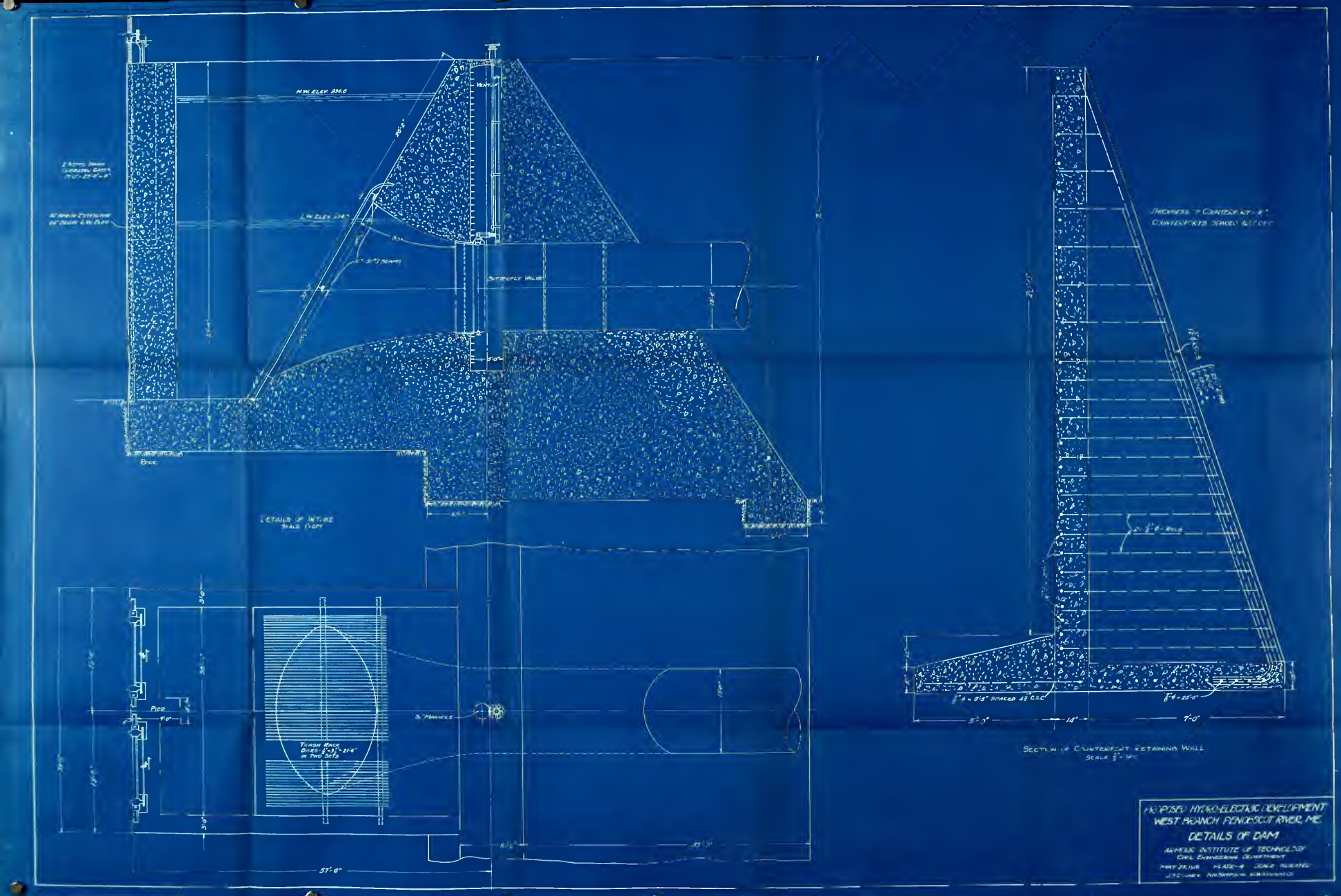


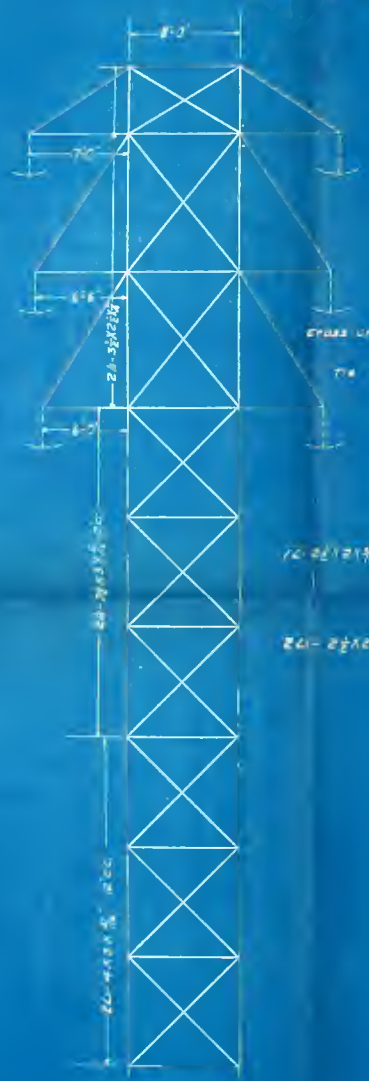
EARTH DAM & CORE WALL
SCALE 1"=8FT.

PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

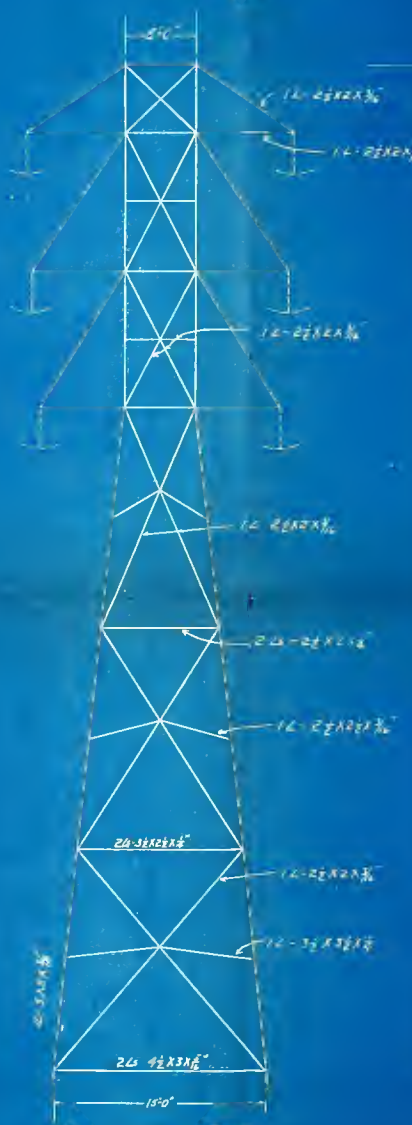
DETAILS OF DAM

MASS. INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1921 PLATE-3 SCALE-INDICATED
J.T. COLLINS H.W. THOMPSON H.W. TWINNELL

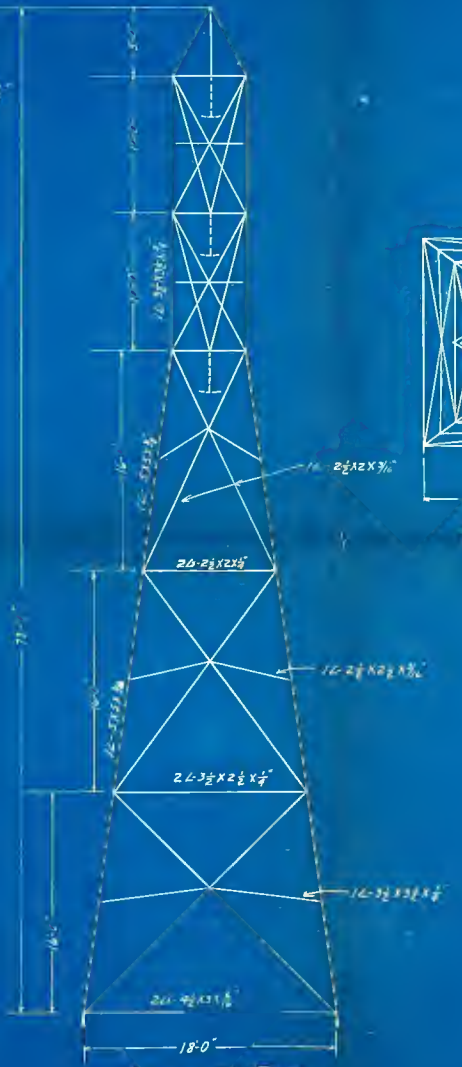

H.W. ELEV. 2040
L.W. ELEV. 2014
Rock
DETAILS OF INTAKE
SCALE 1"=10'
Pier
Trash Rack
Bars 3"x⅜"x 21'6"
in Two Sets
57'-0"
Thickness "T" Counterfort - R"
Counterforts Spaced @ C to C
Section of Counterfort Retaining Wall
Scale ⅜"=1'-0"
KENNEBEC HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME
DETAILS OF DAM
MAINE INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT


8'-0"
7'-0"
6'-0"
2 L - 3½ X 2½ X ½"
cross arms 4 at - 2½ X 2 X ¼"
tie 1 L - 2½ X 2 X ¼"
1 L - 2½ X 2½ X ¼" for all h.v. wires
2 L - 2½ X 2 X ¼" for all h.v. wires
2 L - 4 X 3½ X ½" 12'-0"
2 L - 4½ X 3½ X ½" 12'-0"

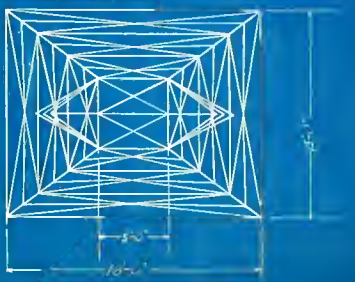
Flexible Tower



Front View Rigid Tower



Side View Rigid Tower

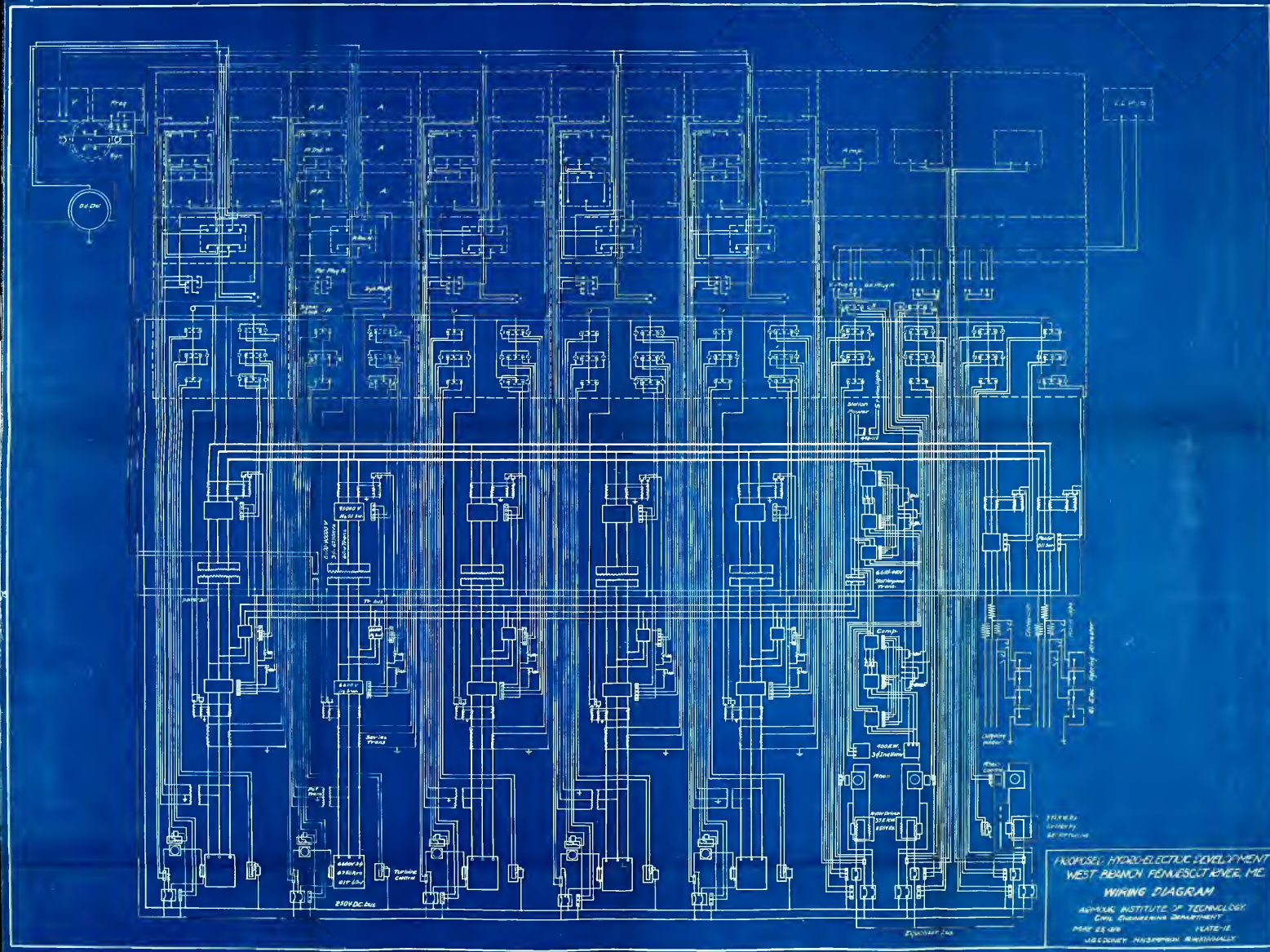


Top view Rigid Tower

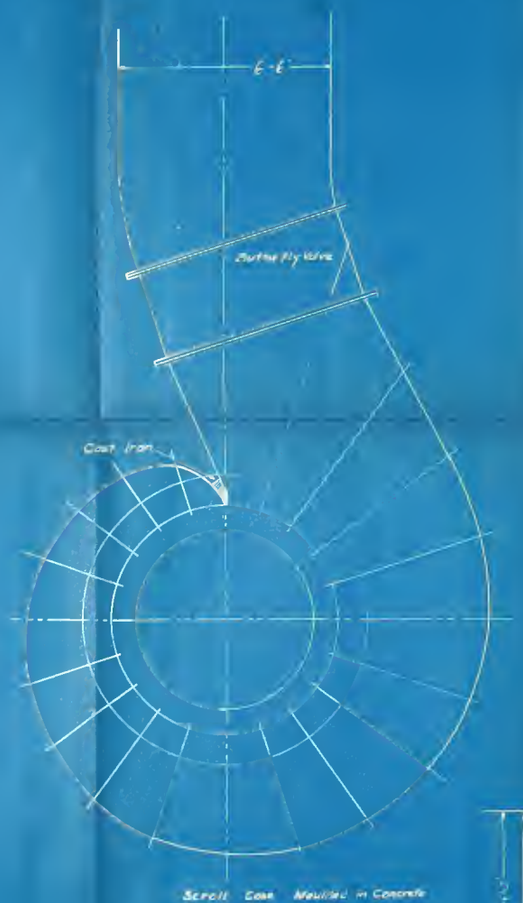
PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME

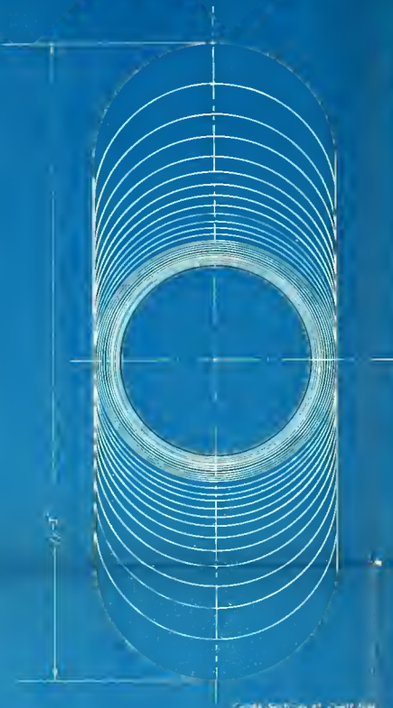
TRANSMISSION TOWERS

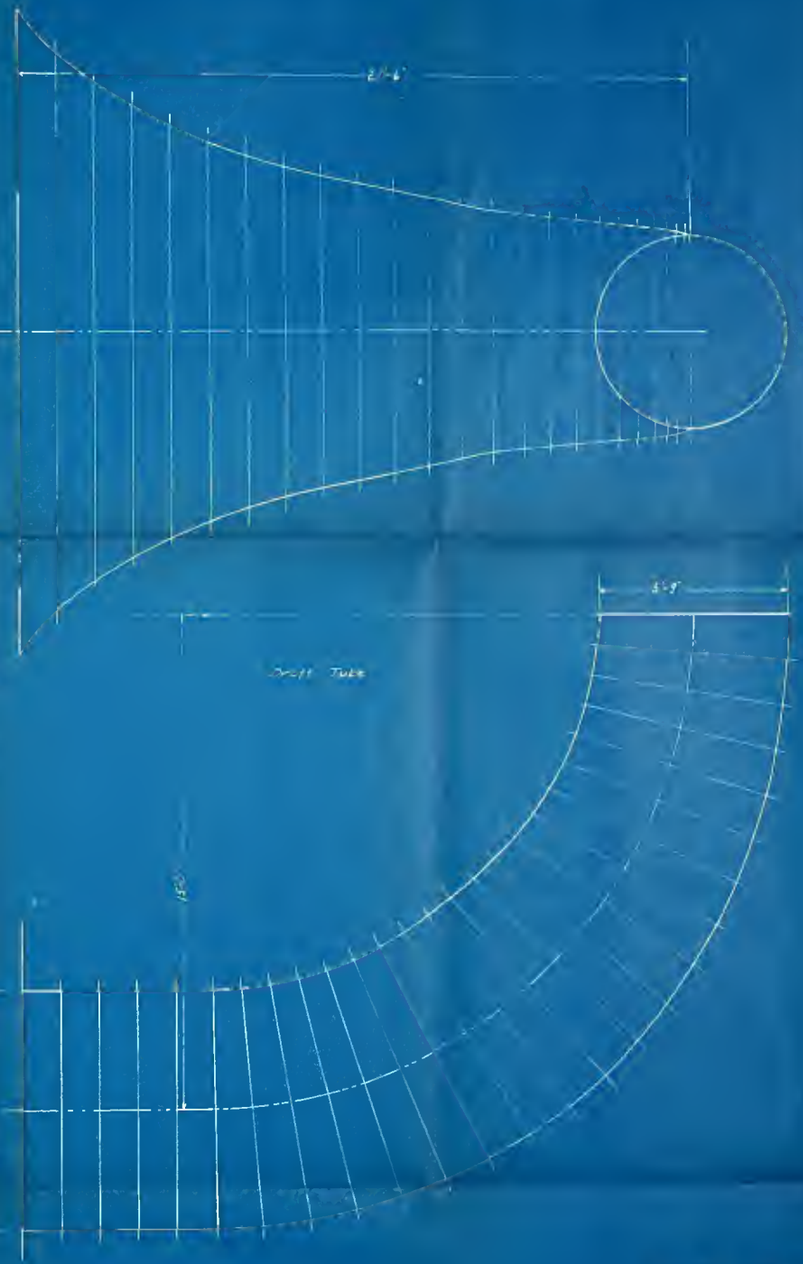
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1911. PLATE 18 SCALE ½"
J.B. CROCKER H.L. SIMPSON, INSTRUCTORS

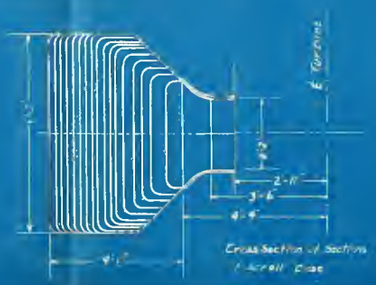


PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENNESCOT RIVER, ME.
WIRING DIAGRAM
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 23 '20 PLATE-12
J.G.COONEY H.J.THOMPSON R.W.WALLACE

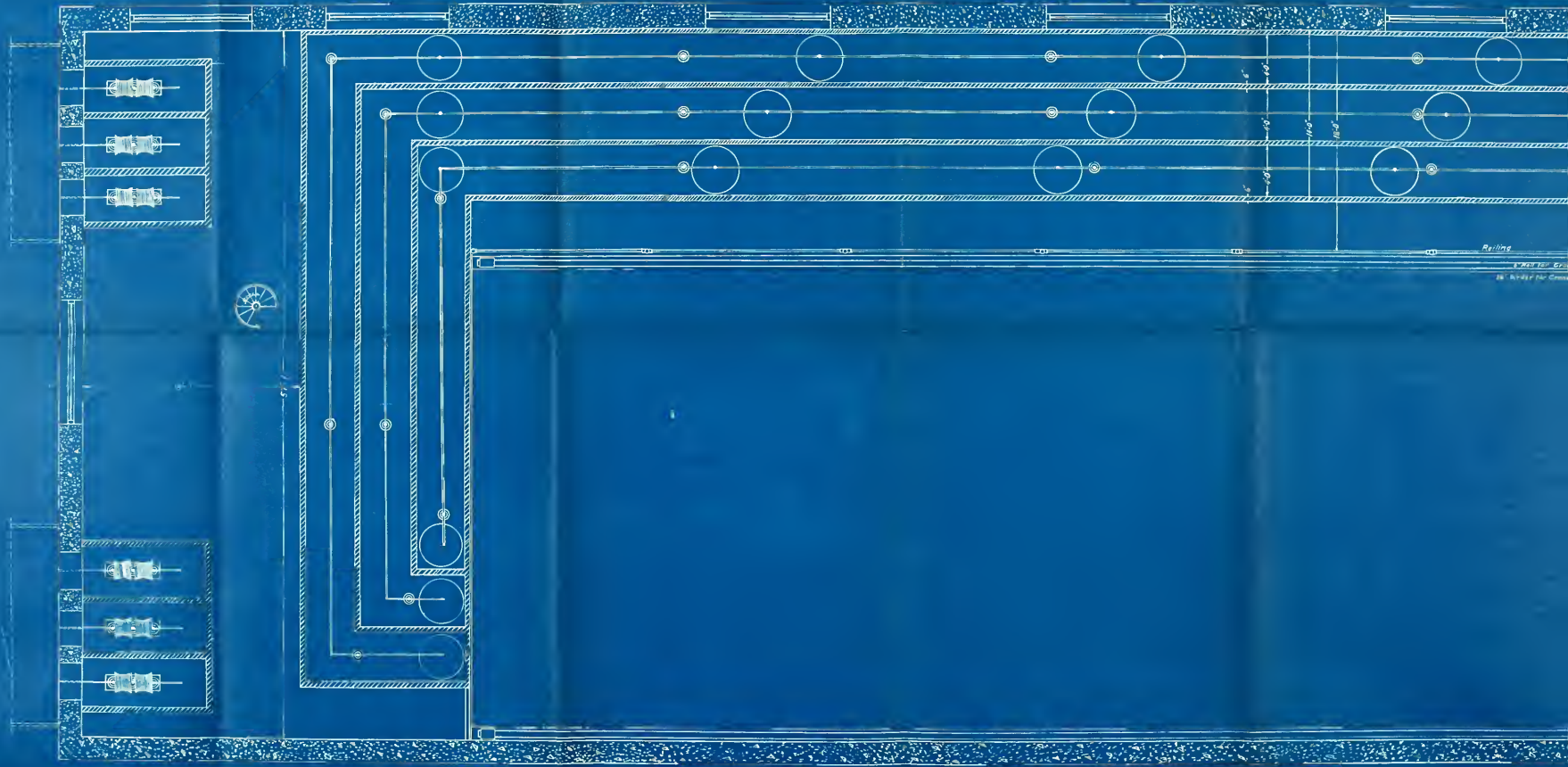


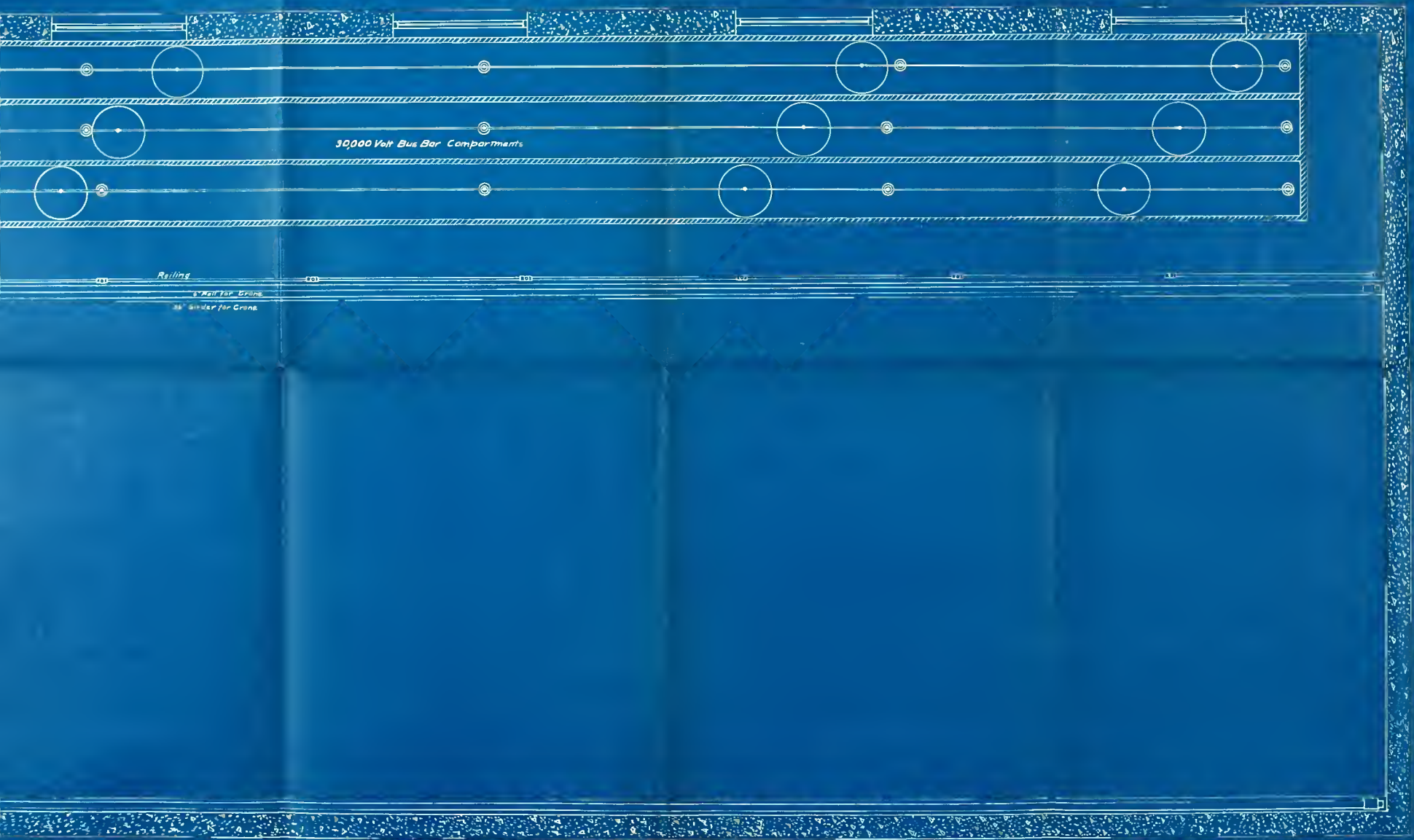






PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.
Design of
SCROLL CASE AND DRAFT TUBE
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 23, 1914 PLATE-D SCALE 1"=1'
INSTRUCTOR IN CHARGE

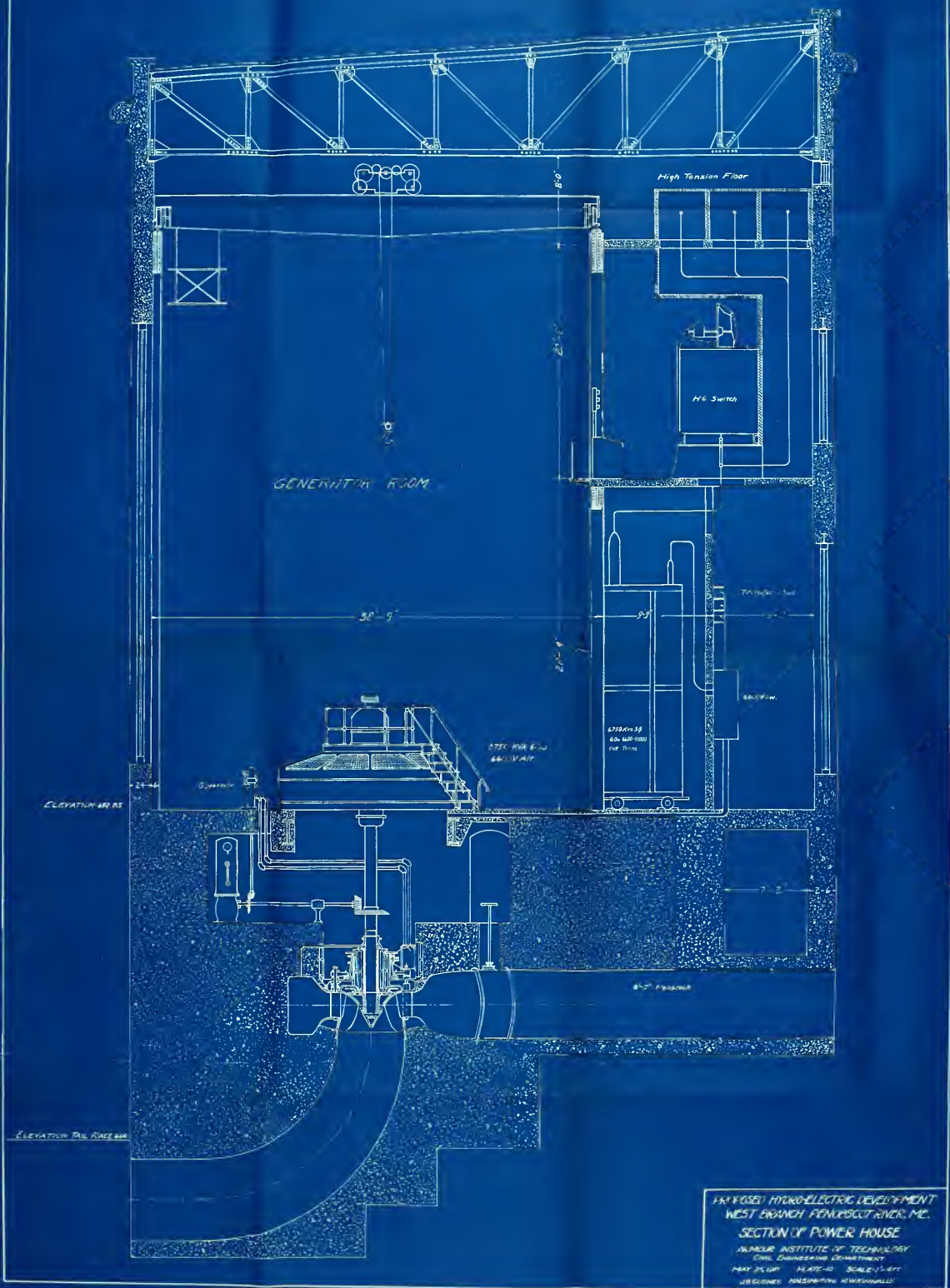

Railing



PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

POWER HOUSE PLAN, 3rd FLOOR

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1916 PLATE 4 SCALE 1/4"
VELLUM SET WASHINGTON ORIGINALS



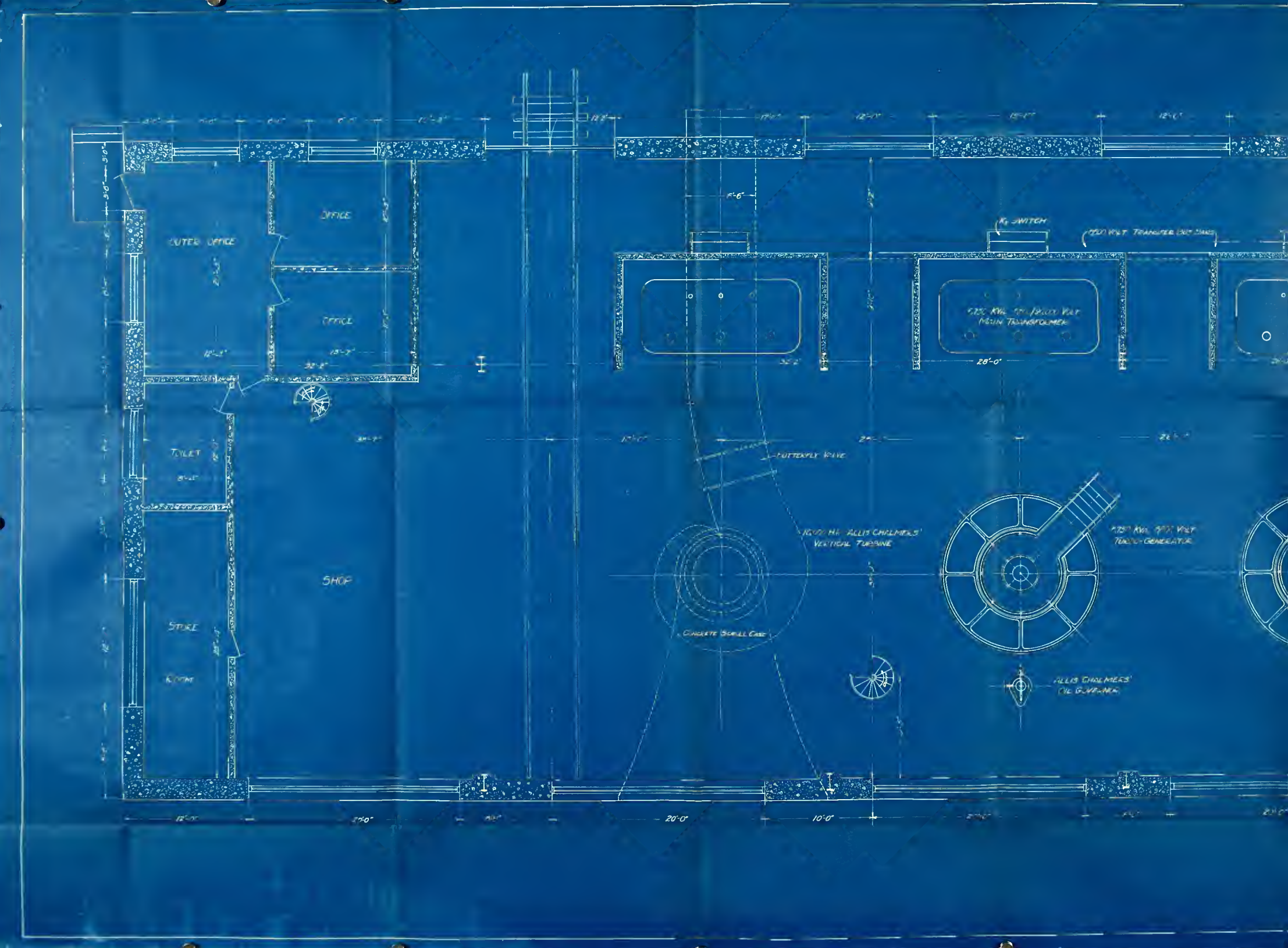
HAVOSEN HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

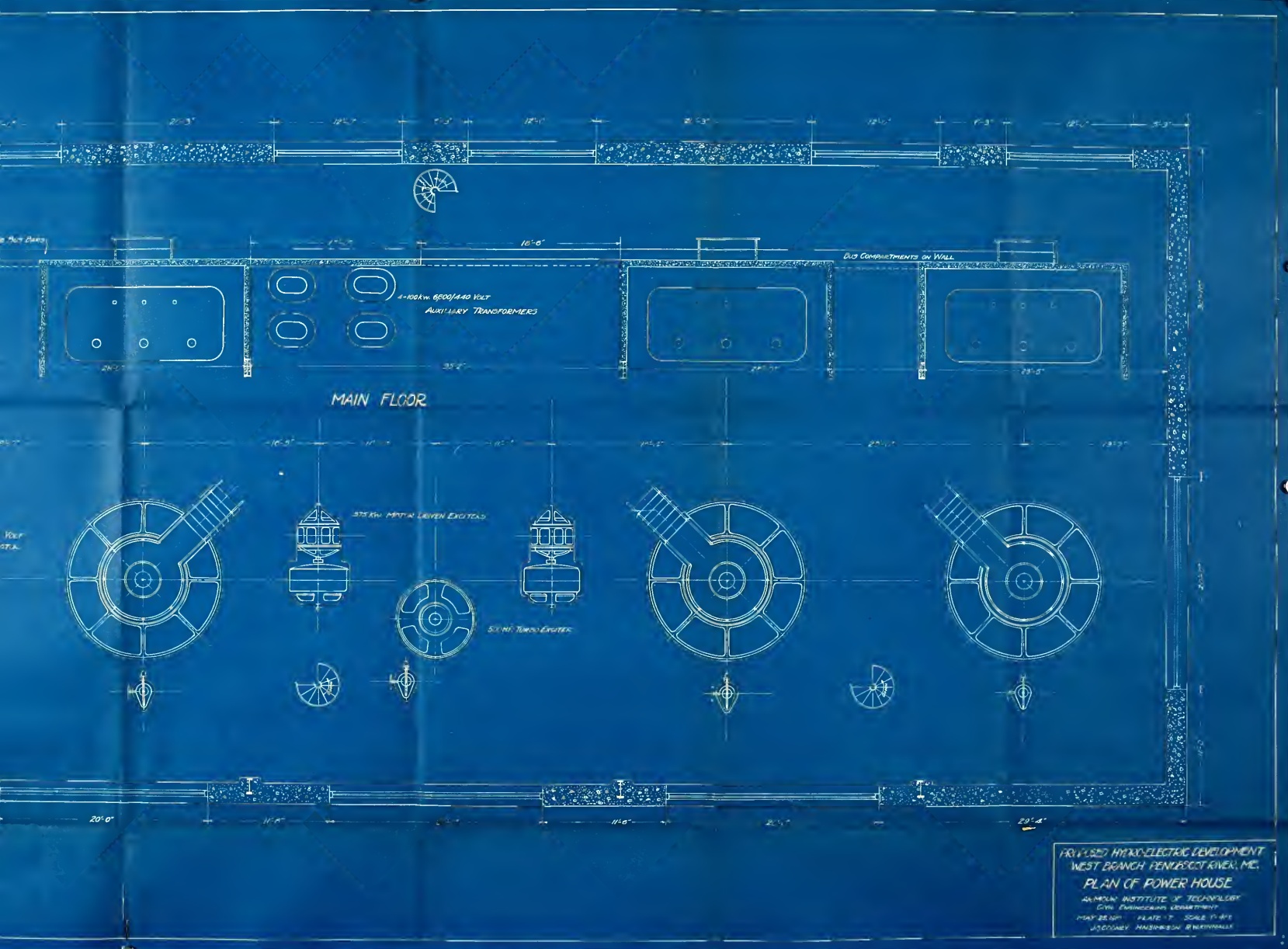
SECTION OF POWER HOUSE

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

MAY 25 1911 PLATE-10 SCALE: 1/4 FT

J.B. CLEMENT MANSFIELD HOWARD HALL


OUTER OFFICE
OFFICE
OFFICE
TOILET
SHOP
STORE
ROOM
5'-6"
K.V. SWITCH
4160 VOLT TRANSFER (OUT DOOR)
4250 KVA 4160/2300 VOLT MAIN TRANSFORMER
28'-0"
BUTTERFLY VALVE
10000 H.P. ALLIS CHALMERS' VERTICAL TURBINE
CONCRETE SCROLL CASE
4250 KVA 4160 VOLT TURBO-GENERATOR
ALLIS CHALMERS' OIL GOVERNOR
20'-0"
10'-0"



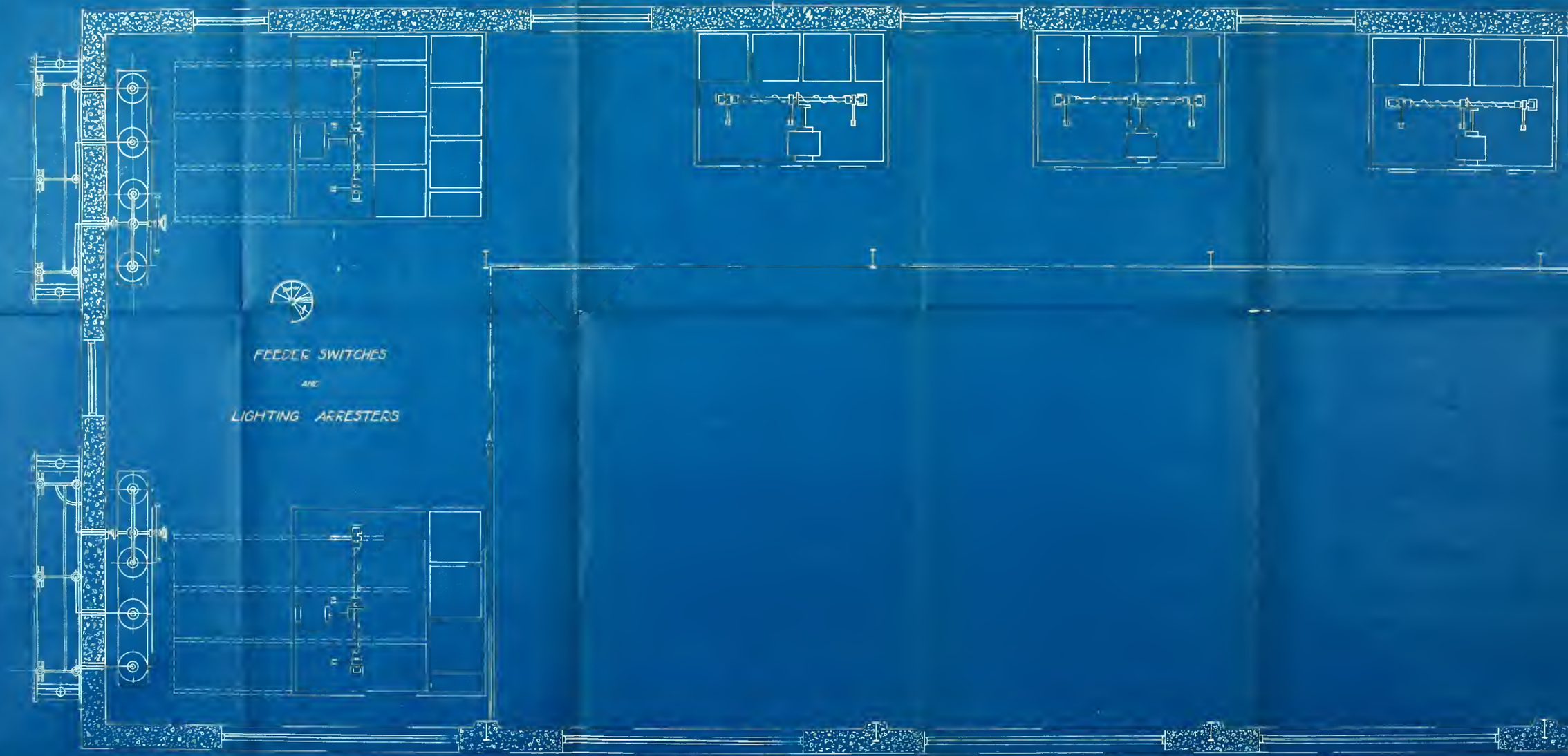
PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

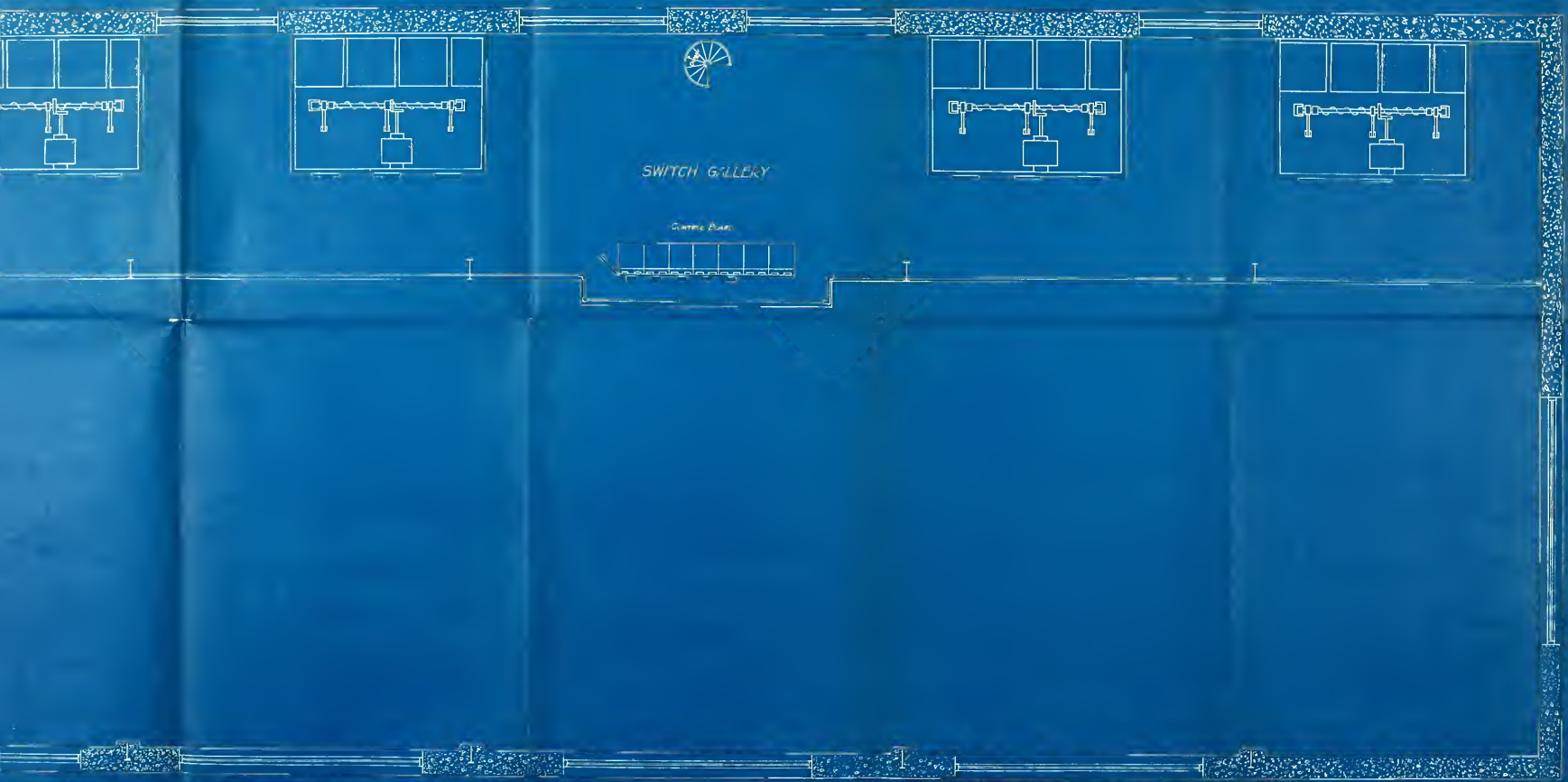
PLAN OF POWER HOUSE

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

MAY 28 1917 PLATE 7 SCALE 1/4"=1'0"

J.T. COONEY HANSON & MERRIMAN


FEEDER SWITCHES
AND
LIGHTING ARRESTERS



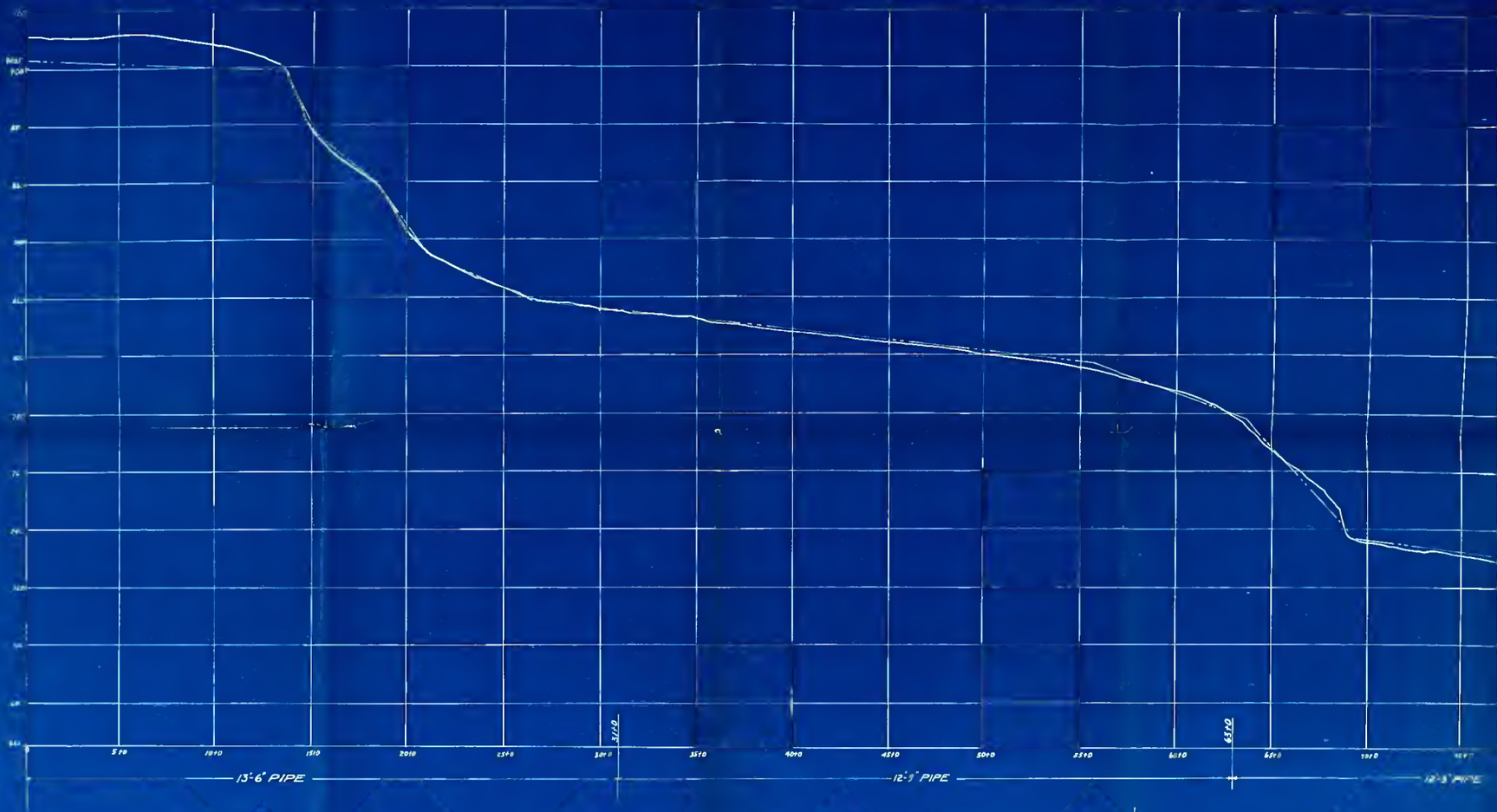
PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

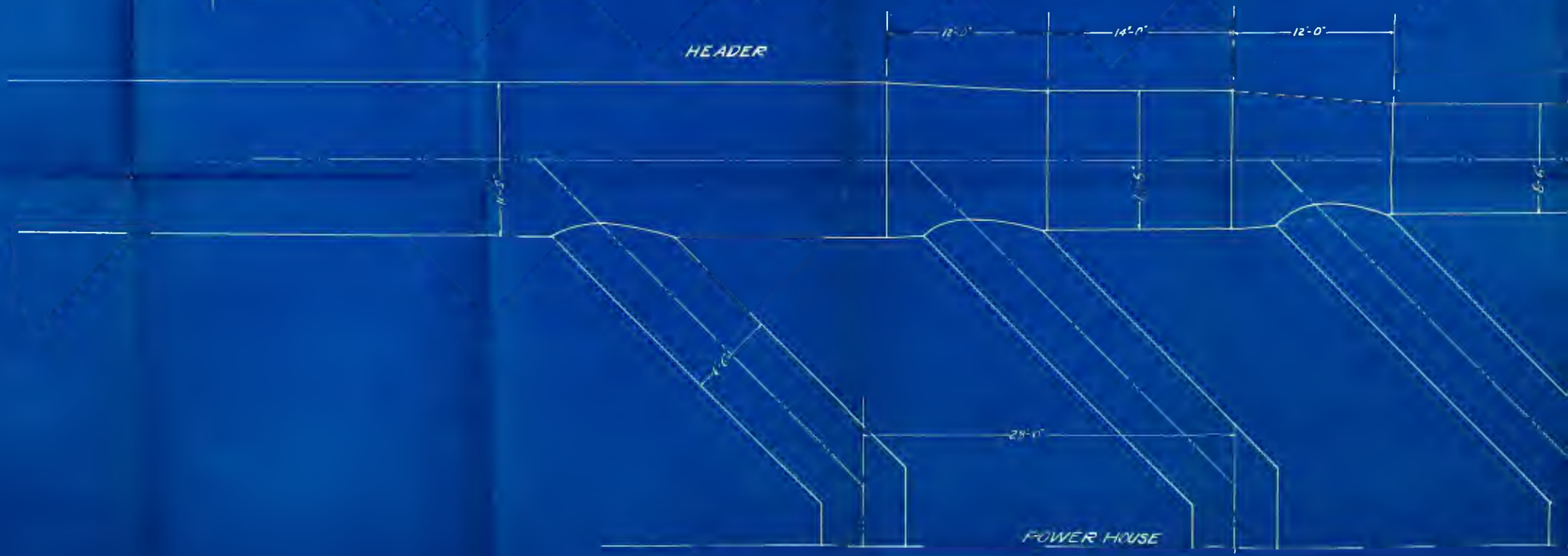
POWER HOUSE PLAN, 2ND FLOOR

MASS. INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

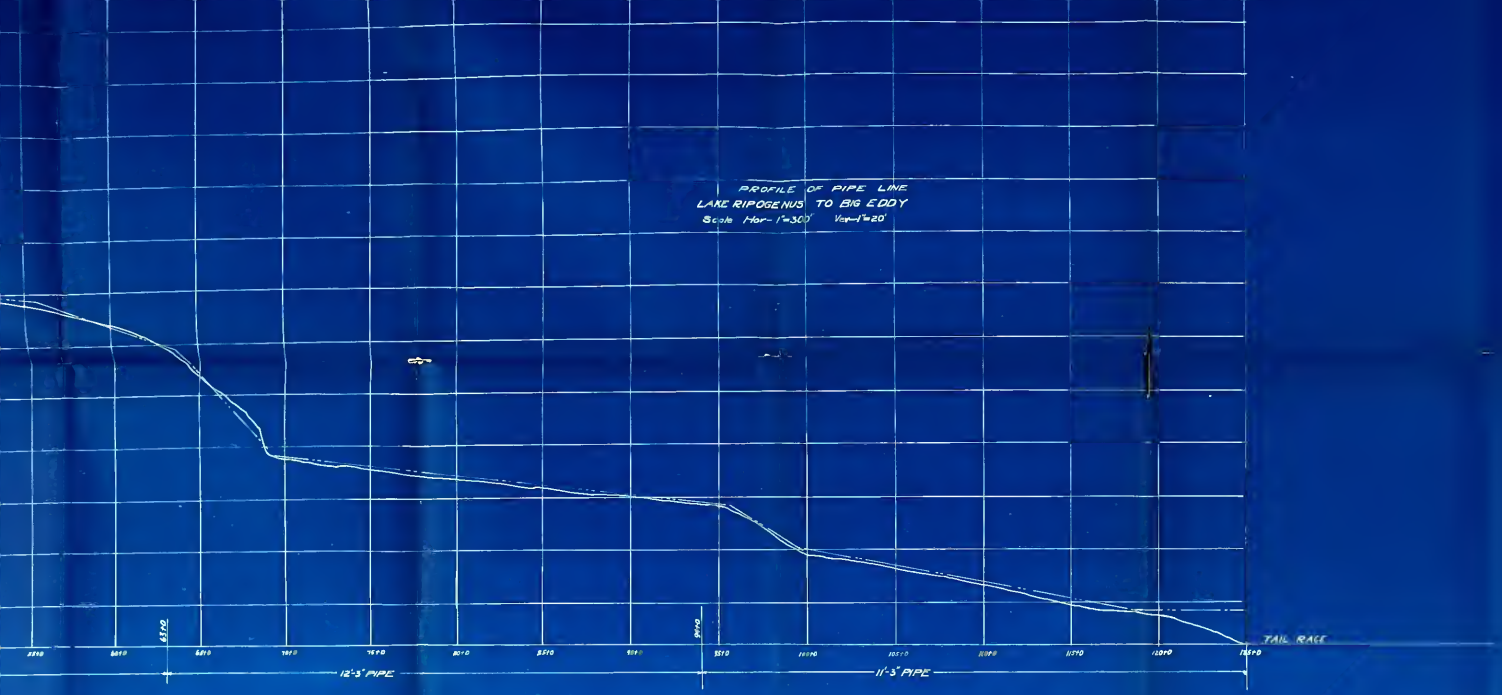
MAY 25, 1918 PLATE 5 SCALE 1/8 FT.

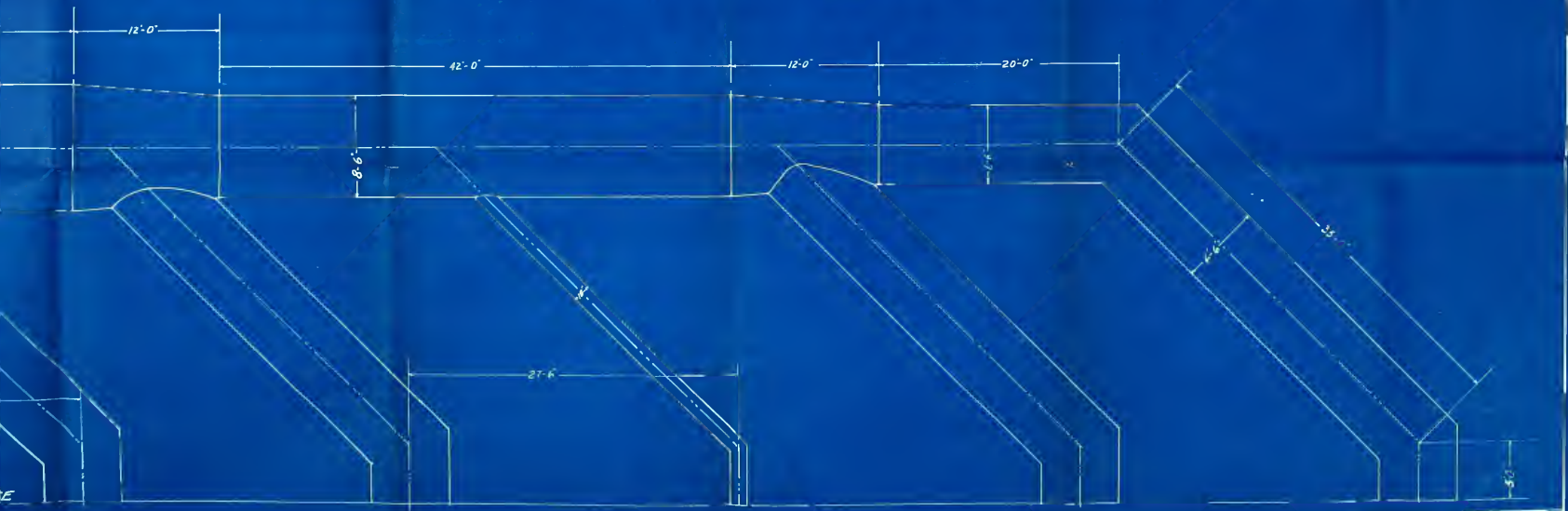
DRAWN BY HALBERTSON, WHITTINGHILL


13'-6" PIPE
12'-9" PIPE
12'-9" PIPE

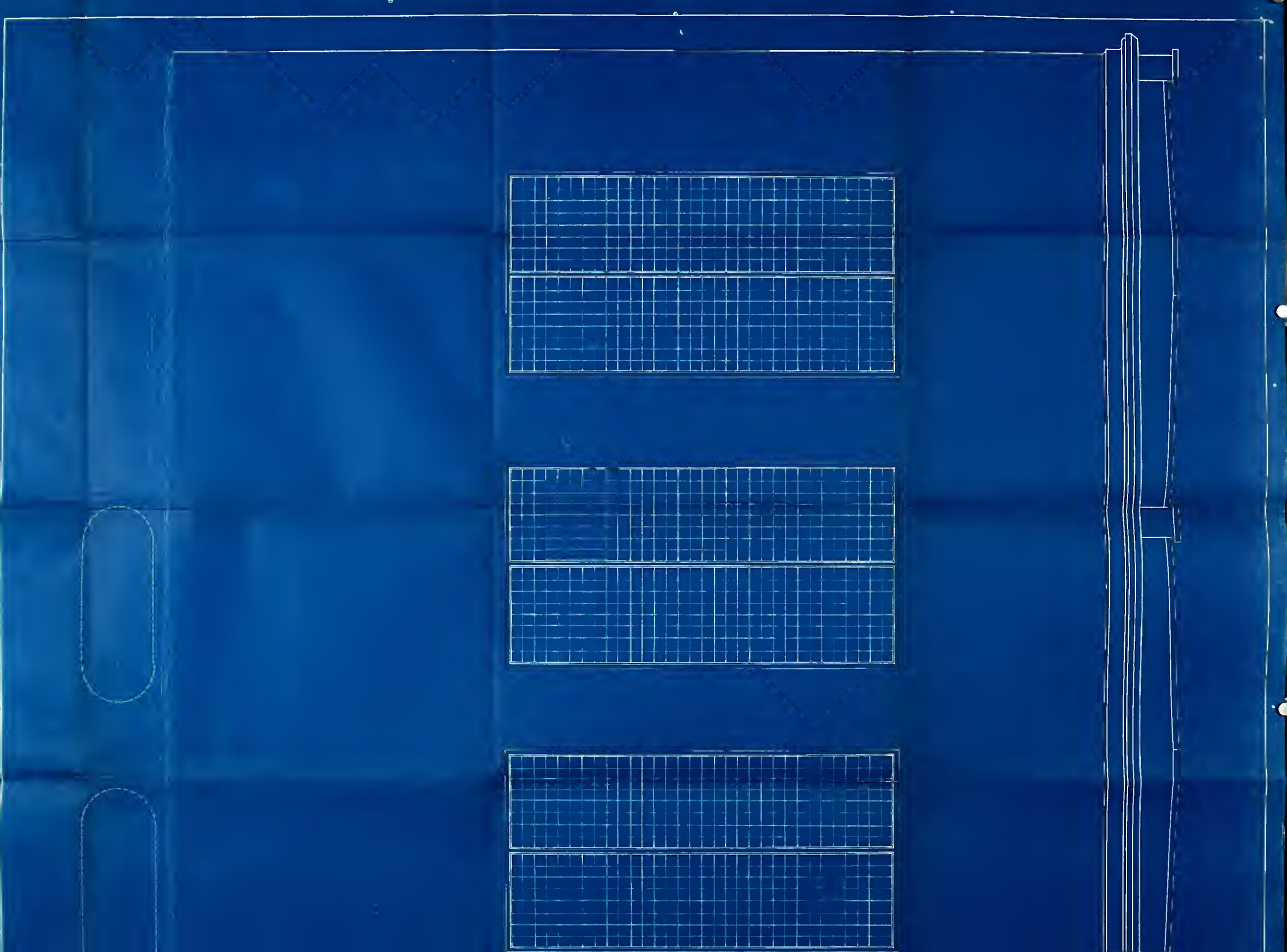

HEADER
12'-0"
14'-0"
12'-0"
28'-0"
POWER HOUSE

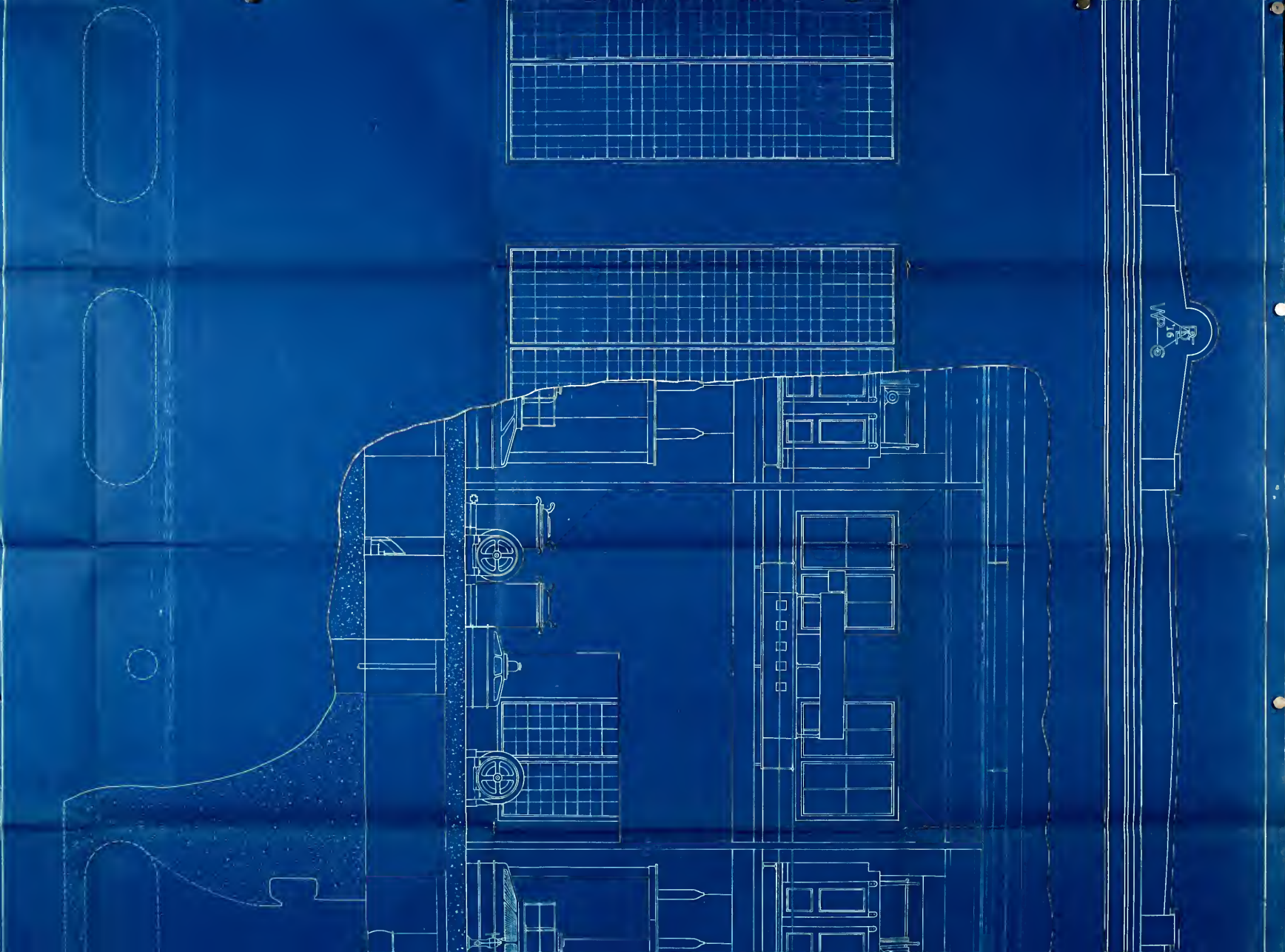
PROFILE OF PIPE LINE
LAKE RIPOGENUS TO BIG EDDY
Scale Hor- 1"=300' Vert-1"=20'

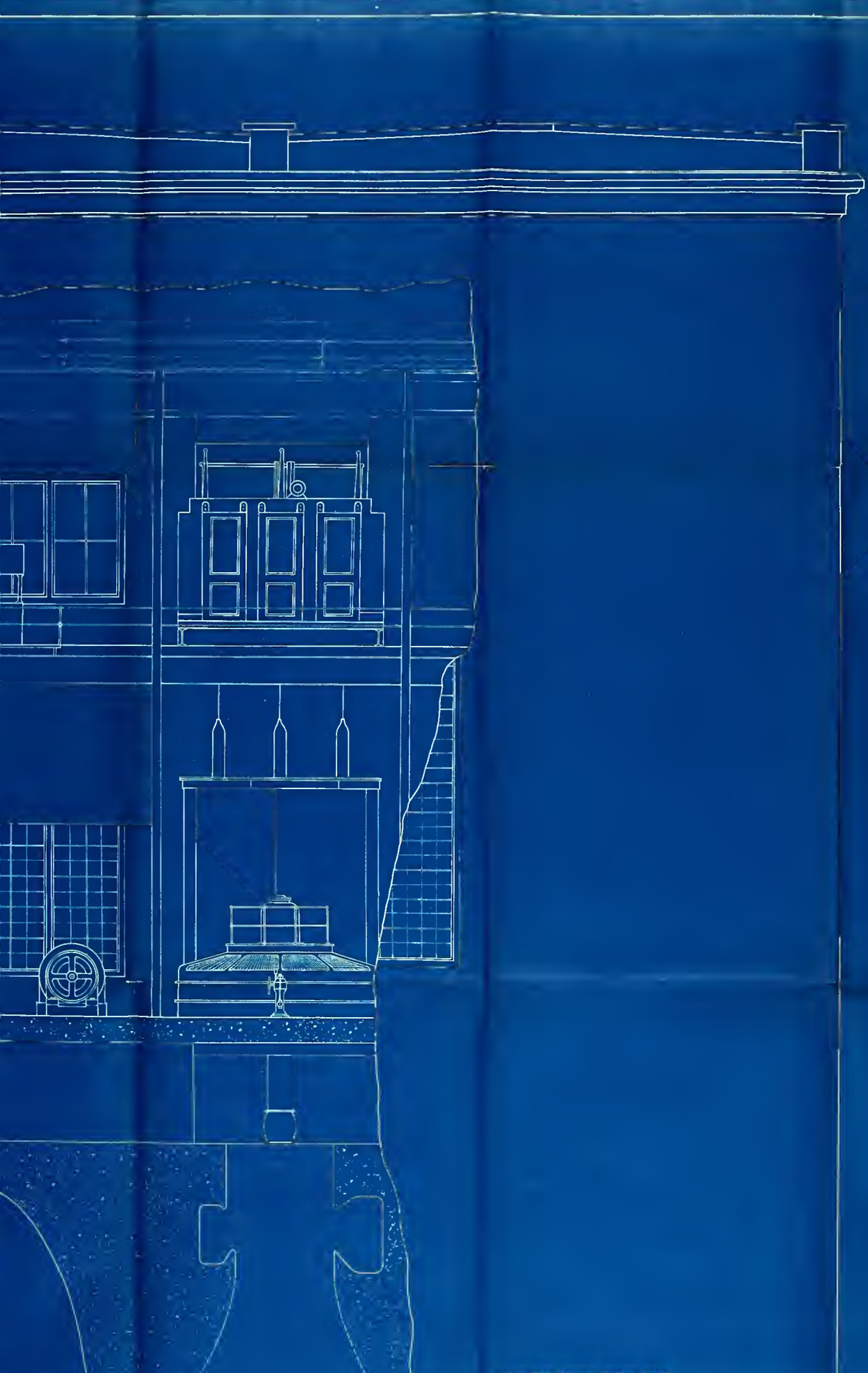




PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.
HEADER AND PROFILE
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
MAY 25 1916 PLATE 5 SCALE-NO.
J.S. COONEY H.N. JEFFERSON E.W. KINNALLY







PROPOSED HYDRO-ELECTRIC DEVELOPMENT
WEST BRANCH PENOBSCOT RIVER, ME.

WATER-FRONT ELEVATION OF POWER HOUSE

MAINE INSTITUTE OF TECHNOLOGY
Civil Engineering Department

DATE 6/5/08 PLATE-II SCALE 1/4"=1'

DRAWN BY [illegible]